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WATERWAYS AND HARBORS DIVISION

Proceedings of the American Society of Civil Engineers

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WATERWAYS AND HARBORS DIVISION

Proceedings of the American Society of Civil Engineers

DEEPENING OF WILSON LOCK ELIMINATES THIRD LOCKAGE

W. F. Emmons,[1] F. ASCE and O. Lavik,[2] F. ASCE

SYNOPSIS

TVA is building a new large single-lift lock at Wilson Dam to replace the present 35-year-lift lock system.[3] To utilize the old system for standby service, one low-lift lock will be abandoned and the lower chamber of the remaining double-lift lock will be deepened 10 feet and provided with a new hydraulic system.

General Considerations

Included in the extensive navigation developments being carried on by TVA in the Wilson Dam area are the repair and modification of the present triple-lift lock system. The general location of Wilson Dam and a schematic profile of the navigation structures on the Tennessee River are shown on Fig. I. An interesting picture indicating the early stages of excavation for the original double-lift lock at Wilson Dam is shown on Fig. II.

The present lock system, as shown on Fig. III, consists of the low 10-foot-lift Lock and Dam No. 1, shown in the foreground just above the bridges; the double-lift lock in the Wilson Dam, toward the top; and the 2-1/2-mile-long Florence Canal connecting these locks. The double-lift lock is shown on Fig. IV, to the left of the artist's depiction of the new large lock now being constructed.

Note: Discussion open until February 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2167 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. WW 3, September, 1959.

1. Head Civ. Eng., Tenn. Val. Authority.
2. Staff Civ. Design Eng., Tenn. Val. Authority.
3. See Proceedings Paper 1069 entitled "Design Consideration for the New Lock at Wilson Dam on the Tennessee River" by R. A. Monroe and George P. Palo.

FIG I

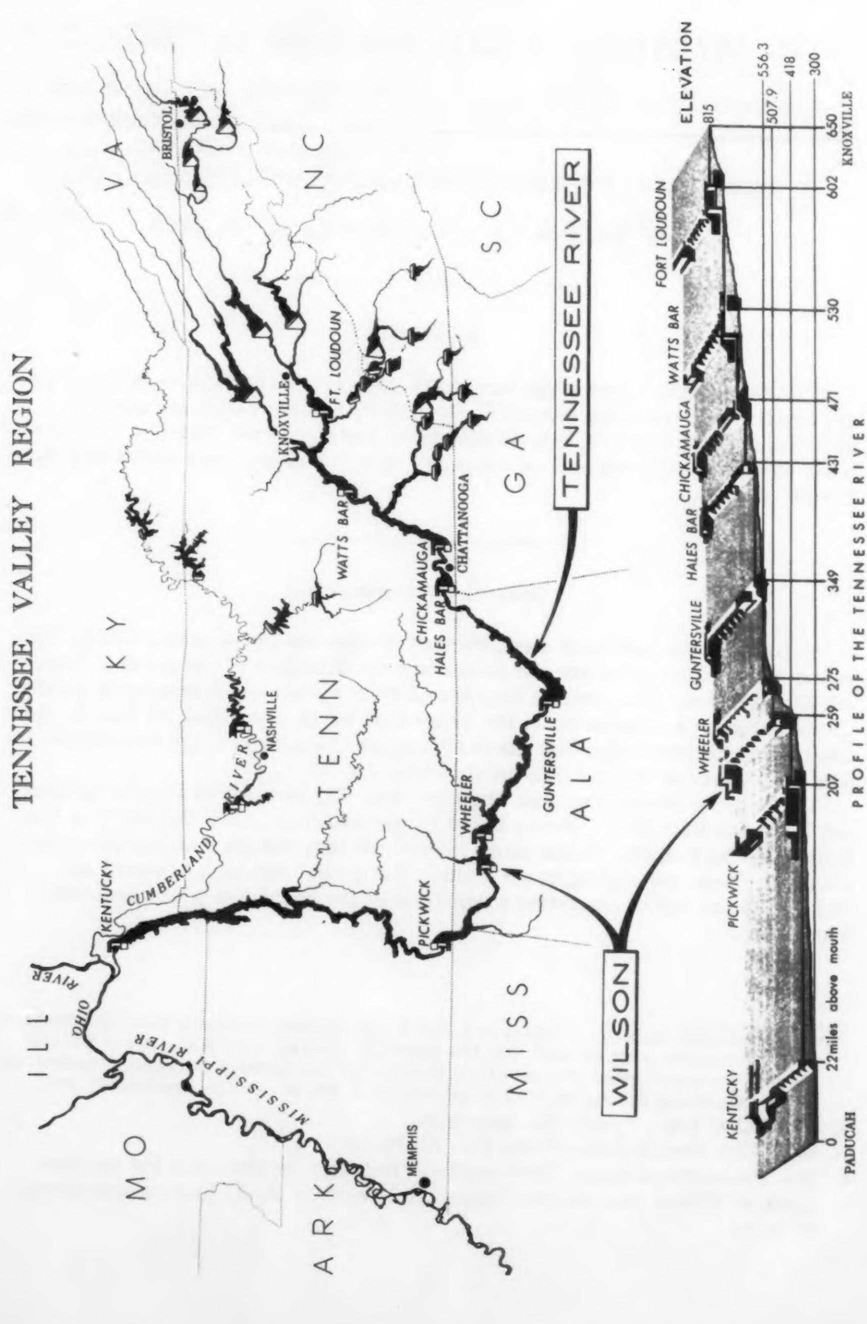

TENNESSEE VALLEY REGION
TENNESSEE RIVER
WILSON
MO
ILL
KY
VA
NC
SC
GA
ALA
MISS
TENN
ARK
MISSISSIPPI RIVER
OHIO RIVER
CUMBERLAND RIVER
KENTUCKY
PICKWICK
WHEELER
GUNTERSVILLE
HALES BAR
CHICKAMAUGA
CHATTANOOGA
WATTS BAR
KNOXVILLE
FT. LOUDOUN
BRISTOL
NASHVILLE
MEMPHIS
ELEVATION
815
556.3
507.9
418
300
KENTUCKY
PICKWICK
WHEELER
GUNTERSVILLE
HALES BAR
CHICKAMAUGA
WATTS BAR
FORT LOUDOUN
0
PADUCAH
22 miles above mouth
207
259
275
349
431
471
530
602
650
KNOXVILLE
PROFILE OF THE TENNESSEE RIVER

FIG. II

FIG III

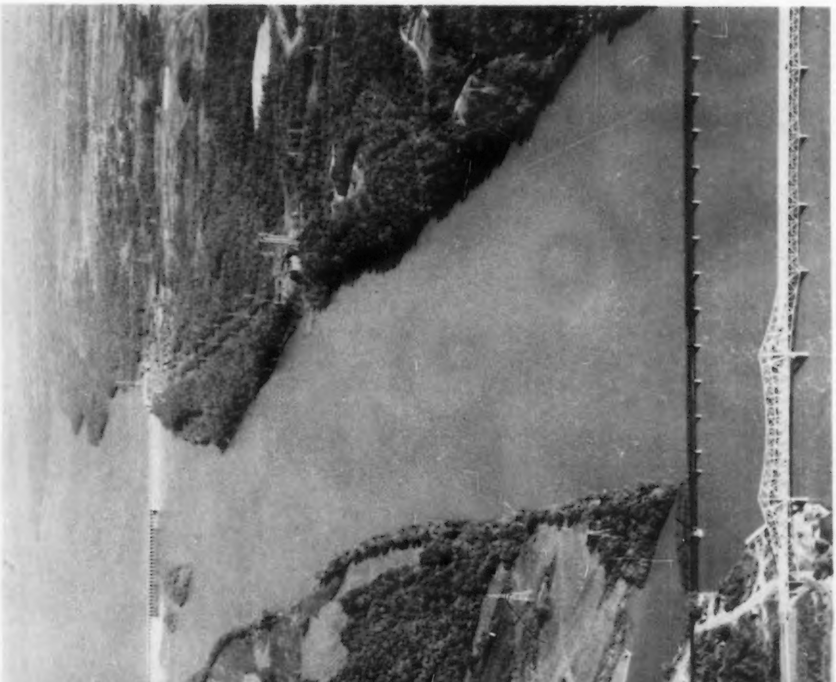
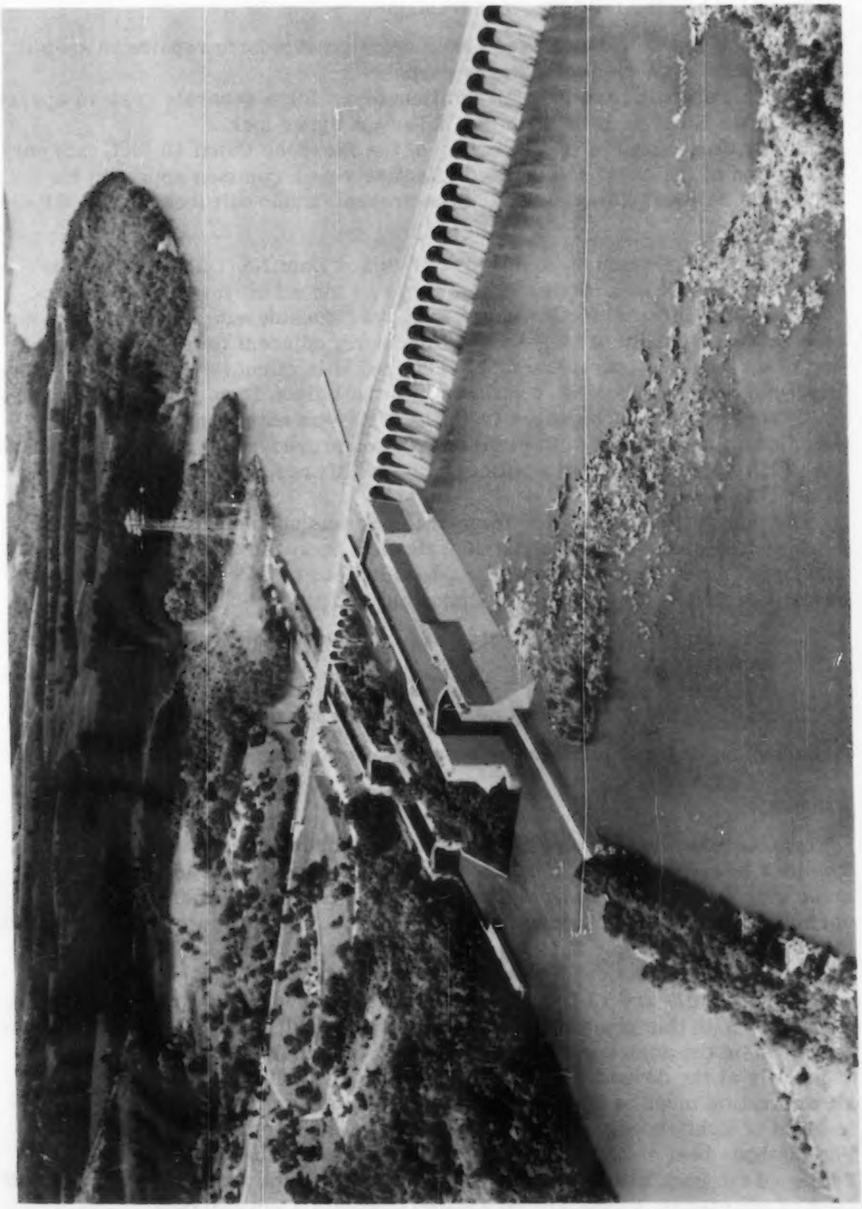


FIG IV



Because of the heavy traffic at Wilson, it was planned to provide standby facilities for use when the main lock is out of operation for inspection and maintenance. The use of the present lock system for this purpose was considered but had the following disadvantages:

1. Lock and Dam No. 1 would have required extensive repairs to keep it operational even for standby purposes.
2. There would have been an additional cost for a separate crew to operate Lock No. 1 as it is 2.5 miles below the upper lock.
3. By deepening the upper portion of the Florence Canal 10 feet, this portion of the canal can be used effectively as a common approach for both the new large single-lift and the present double-lift locks at the Wilson Dam.

It was decided, therefore, to abandon Lock and Dam No. 1 and to alter the upper double-lift lock so that it will serve as the standby.

Two plans for altering the upper lock were considered. The first plan was to construct a third chamber with a 10-foot lift adjacent to the downstream end of the existing lower chamber. This would in effect be equivalent to moving the abandoned Lock No. 1 upstream to the Wilson Dam. In reviewing the early construction photographs (see Fig. II) it was noted that vertical rock faces could be obtained with usual excavation procedures. This suggested the possibility of obtaining the additional 10-foot lift required by deepening the lower chamber.

The plan for deepening the lower chamber was adopted as the estimated construction cost was less and a considerable increase in the traffic capacity will result from there being two lock lifts instead of three. This plan posed several problems, with the more important being:

1. Excacation of rock in the chamber between the lock walls without causing damage to these walls.
2. Design of a satisfactory and economical hydraulic system.
3. Modification of the lower miter gate to serve the deepened chamber.

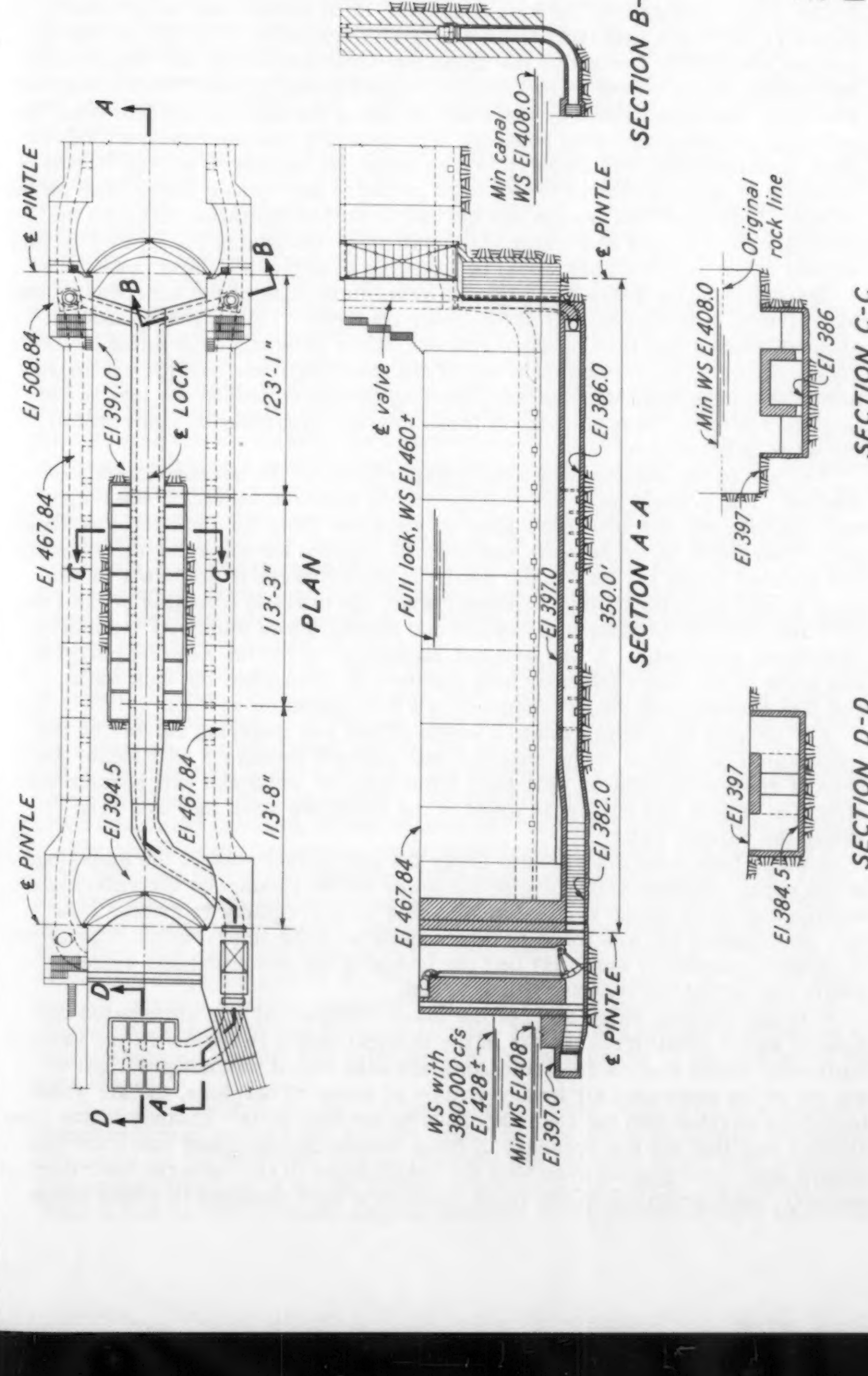
A discussion of these problems and proposed solutions follow.

Excavation

Typical plan and sections of the lower lock chamber are shown on Fig. V. The lock is set in a formation of horizontally bedded cherty and brittle limestone with thin shale partings. This formation is cut by numerous nearly vertical joints, and during blasting there is a decided tendency for the horizontal partings to open up. This sometimes results in horizontal and vertical movements at a considerable distance from the blast.

The lock walls are 15 feet thick by about 60 feet high and are poured against the rock. With this proportion of thickness to height, it is evident that we have to depend on the adjacent rock for stability. The rock along the river wall, especially at the downstream end, is badly broken and jointed. Consequently, all excavation must be done very carefully. To accomplish this work, a combination of light shooting, jacking, and bullpointing is expected to be used. The methods best suited will have to be developed as the work progresses. Facing of the rock below the base of the walls is not planned at the present. If deemed necessary, gunite will be applied to obtain a smooth face.

FIG V


PLAN
℄ PINTLE
℄ PINTLE
El 508.84
El 397.0
℄ LOCK
El 467.84
El 394.5
El 467.84
123'-1"
113'-3"
113'-8"
SECTION A-A
Full lock, WS El 460±
℄ valve
℄ PINTLE
℄ PINTLE
El 386.0
El 397.0
El 382.0
350.0'
El 467.84
WS with 380,000 cfs El 428±
Min WS El 408
El 397.0
SECTION B-B
Min canal WS El 408.0
SECTION C-C
Original rock line
Min WS El 408.0
El 386
El 397
SECTION D-D
El 397
El 384.5

The Hydraulic System for the Lower Chamber

The existing and proposed systems are shown on Fig. V. The existing system consists of wall culverts and ports with the necessary controls. There is one 7- by 9-foot culvert in each wall, and each culvert has twelve 2.5- by 3-foot ports opening into the lock chamber. Each culvert has three 6-inch diameter air vents extending to the top of the lock walls. The filling valves are set about 34.5 feet above the minimum water surface in the lower chamber below. Spool valves were installed originally and failed. The vacuum and resulting vibration caused by the water dropping through the vertical shaft resulted in cracking the valve housing. Hydraulically operated cylinder valves were installed next, and a restriction as shown on section A-A, Fig. V, was installed in each culvert to create back pressure and reduce the vacuum on the valves. This modified system worked and is still in operation although the turbulence in the lock is greater than desirable. Deepening the lock by 10 feet without other modifications would be expected to aggravate these conditions.

Inspection some years ago revealed severe cracking of the concrete around the culverts in the river wall of the lower chamber. This cracking and consequent leakage has been of some concern, and a program of grouting of the rock adjacent to the downstream end of the river wall was carried out shortly after the cracks were discovered. Blasting needed to deepen the lock would be expected to increase the size of these cracks regardless of how carefully the work is performed.

The use of the existing hydraulic system to serve the deepened lock was studied. The present wall culverts have their floors at elevation 408.8 and this is above the new low water level of elevation 408.0 for the deepened chamber. Therefore, these culverts could not be utilized for emptying the chamber, and because of the air they would entrap at each lockage it would not be feasible to use them for filling purposes. Also, the cracked condition of the river wall and rock would make any repairs and modifications required extremely hazardous and costly. It was decided, therefore, to fill the existing culverts and ports of the lower chamber with concrete to strengthen the lock walls, and this necessitated the development of a new hydraulic system.

For the new hydraulic system a single filling and emptying culvert on the longitudinal centerline of the chamber was adopted because it will locate the area of deepest excavation well away from the lock walls and will require construction of only one culvert in place of the usual two when they are at or within the culvert walls.

It would have been desirable to lower the valves that control the emptying of the upper chamber and filling of the lower as the vacuum on the valves and trouble due to air in the vertical shaft would be appreciably reduced. This was investigated but was found to be so costly as to be impractical. Therefore, it became practically essential that the layout of the new hydraulic system retain and utilize the existing filling valves.

With the lowered water level of the lower chamber, it was anticipated that trouble would result from the air in the vertical shafts below the filling valves. Hydraulic model studies confirmed this and also that it was not practical to get rid of the entrapped air by any system of vents. Therefore, the air would have been carried into the chamber, causing serious boils. Another factor considered was that the low pressure at these valves during filling operation had always been troublesome even with the restrictions in the culverts described earlier. Back pressure on the valve could have been obtained by sharp bends,

local restrictions, or by reducing the area of the shaft. Sharp bends and local restrictions would have resulted in areas of low pressures with danger of cavitation, but reduction of the area of the shaft would not have this disadvantage and, in addition, would reduce the volume of entrapped air. Therefore, the vertical shaft was reduced from a 7- by 9-foot rectangular to a 5-foot circular opening. These two circular conduits, with a total area of 39 square feet, will then merge into the main culvert, which has an area of 102 square feet. Hydraulic model studies were used to help develop these dimensions and indicate that this will create sufficient back pressure to prevent serious cavitation to the valve.

The spacing and size of the culvert ports were proportioned from TVA locks having wall ports. These ports will discharge into trenches, one on each side of the culvert. To reduce the amount of trenching, the ports were originally located at the roof of the culvert, but hydraulic laboratory tests were disappointing. All the entrapped air was carried into the chamber through the upstream ports during the first few seconds after the valves were opened, resulting in considerable turbulence. After the air had escaped, the flow through the various ports was unequal and the water tended to flow straight up from the ports instead of impinging on the walls of the trenches and spreading out uniformly. To improve this condition, the ports were moved to the floor of the culvert, and baffle walls were installed between ports. The results were gratifying as the air entered the chamber, spread evenly over the length of the culvert, and it took a much longer time for the air to escape into the chamber. A large percentage of the air evidently was trapped in the culvert, and in the model it was observed to escape through the valve shaft during the ensuing emptying operation. The deeper setting of the ports and the baffle walls resulted in a good distribution of the flow into the chamber and greatly reduced the turbulence. Some modification of the port spacing and sizes was also made in the Hydraulic Laboratory to reduce the longitudinal surges and high hawser stresses indicated by the model.

The downstream gate blocks of the river wall are badly cracked and in need of repair. During closing of the gate these blocks deflect to such an extent that the gate leaves seat at the top while there is still a considerable gap at the waterline. As mentioned before, the rock backing is also badly broken and if the gate blocks were retained the future safety of this part of the structure would have been questionable. Therefore, with repairs difficult and no great quantities involved, it was decided to completely rebuild these blocks.

The locks on the Tennessee River have duplicate filling and emptying systems with one filling and emptying valve and connecting culvert in the river and land wall. This arrangement permits the valves in one wall to be shut down for maintenance without necessitating taking the lock out of service as the filling and emptying system in the other wall can continue to function. Because this deepened lock at Wilson will be a standby facility, it was decided that for this use a single emptying valve and culvert would be satisfactory. Therefore, advantage was taken to simplify the outlet system by laying out a single emptying culvert and valve system for installation in the new river wall gate blocks.

The single emptying valve, 10 feet wide by 10 feet high, will be a conventional reversed radial gate, and the design of this posed no special problems. The valve on the land wall will be removed and the shaft filled with concrete. The outlet structure, a single lateral with side ports, works reasonably well. Boils a foot or two in height may be expected in the prototype right above the

ports. However, these die out in a short distance downstream, and considering the standby nature of these facilities it was felt further refinement at an increase in cost was justified.

The new hydraulic system will fill or empty the chamber in 14 to 15 minutes, depending on pool elevation and the valve speed finally set for operation.

The present lower chamber cannot be filled completely by emptying the upper chamber. To compensate for this, the filling valves of the upper chamber are kept open for some time while the lower chamber is filling. The deepening of the chamber will increase this overlapping time interval but otherwise should not change the operation.

Lower Miter Gate

Each leaf of the existing lower miter gate weighs 170 tons. They will have to be moved out of position to allow excavation and rebuilding of the gate blocks of the river wall and extension of the quoin on the land wall. A 10-foot extension will be added to the bottom of these gate leaves to provide for the deepened chamber.

The present gate machinery is operated by compressed air and is in need of repair. Because of the very limited use anticipated, a study is in progress to determine the feasibility of repairing and modifying this machinery so it can be utilized for operation of the deeper and heavier gate leaves.

Other Considerations

The emergency needle dam now used for unwatering the lock can be continued in service by a slight modification. The existing towing track and mooring facilities will continue in use, but it will be necessary to deepen the wells 10 feet for the floating mooring bits. No alteration to the upper chamber is planned, but maintenance work will be performed during the outage for the deepening of the lock.

CONCLUSIONS

The construction work will start in the spring of 1959 and is scheduled to be finished in the early spring of 1960. The cost of this work is estimated at $1,500,000 and should keep the 35-year-old lock available for navigation use for many years to come.

The design for the lock modification is being performed by the TVA Division of Design under the direction of R. A. Monroe, M. ASCE, Chief Design Engineer; George P. Palo, M. ASCE, Assistant Chief Engineer; and George K. Leonard, M. ASCE, Chief Engineer.

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WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

ENGINEERING ASPECTS OF COASTAL SEDIMENT MOVEMENT

Richard Silvester[1]

SYNOPSIS

Current knowledge of foreshore processes are summarised illustrating, among other things, the significance of: wave spectra, mass transport, beach water tables and climatological conditions on the cycles of beach erosion and accretion. The importance of the predominant swell in sediment transport and coastal physiography is stressed. The long term effects of groynes and sea-walls located on receding coastlines are discussed. The subject of tidal action upon ocean waves is dealt with in association with the effect of density currents in estuarine conditions. Certain conclusions are presented respecting scale models of coastal sediment movement.

INTRODUCTION

Foreshore processes have been described recently by Bascom in which the latest developments in theory and observation up to 1953(1) were included. Previous to this Inman,(2) Johnson(3) and Eaton(4) had published comprehensive summaries of all factors involved in littoral drift phenomena. But it seems opportune to review these factors again, bringing them up to date, simplifying them, and emphasising those which are of greatest concern to the engineers responsible for erosion mitigation and siltation control.

Theories regarding the movement of beach material have often been at variance, due, in many cases, to the limited scope of the observations made. Too intensive a study of a small section of coast during a short period(5) can create a myoptic view of the complete physiographical unit affecting the sediment movement, which may be hundreds of miles in length.

Also many authorities are discouraged from approaching a coastal problem scientifically by the apparent abundance of data required to arrive at a solution.

Note: Discussion open until February 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2168 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 9, No. WW 3, September, 1959.

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They therefore are swayed by the opinions of the local community or undertake expensive empirical methods of construction.

There thus appears to be a definite need to reduce the data required and to ease the problems of interpretation. One example is that of wave recording where many difficulties can arise in measurement and analysis.(6)

On the other hand, some apparently important measurements, such as wave direction and sediment transport on the ocean bed, cannot yet be made due to the lack of instruments. Nevertheless, sufficient is known of the general beach processes for engineers to be assisted greatly in the location of their maritime structures.

Generation of Waves

The multitude of publications on wind generated ocean waves, recently summarized by Saville,(7) and the differences inherent in the various methods for hindcasting ocean waves, is very disconcerting to an engineer who has had little or no previous contact with oceanography. In dealing with sediment movement along the coast such finesse in computing wave characteristics can be omitted. Although maximum or other values of wave height and period may be necessary for the design of maritime structures, they are not required to assess the long-term process of sediment transport along the coast.

The wave data required are the wave direction and the types of wave encountered. These two distinct types are storm waves and swell. The latter is a well-known and accepted term,(8) but the former has not previously been well defined, it is to receive a strict connotation in this article as will be seen below.

(a) Storm Waves

Where a wind blows in more or less a constant direction for a period of time across an area of ocean, waves will be generated and the area is termed a fetch. Whilst the waves move within this area and are still being built up by the wind they are to be termed "storm waves". Hence for storm waves to reach a coastline the fetch must be adjacent or very close to the coastline in question.

Ocean waves in general do not consist of a sinusoidal train of undulations with similar wave lengths and constant height, but are made up of many trains of different wave length (period) with heights peculiar to each. If these trains were isolated and the height of each graphed against its period the resultant figure would be a spectrum of the waves present as in Fig. 1.(9) The width of the spectrum is a measure of the number of wave trains present. Storm waves have a wide spectrum with one particular train having the largest wave height, the period of which is determined, amongst other things, by the wind velocity, the length of the fetch and the duration of the wind.

It has been shown experimentally(10) and mathematically(11) that waves within a fetch are generated in several directions at once. Arthur(12) discussed the significance of this in 1949 and recently Saville(13) has shown how the width of the fetch affects the multidirectional nature of these storm waves. Fig. 2 depicts the wave crest pattern to be expected within a fetch.(10) It should be realized that such a rectangular fetch is an extreme simplification. Within a cyclonic centre several fetches are created simultaneously, or consecutively, all of which are in motion. Thus the waves in any one region are a composite of those from many areas.

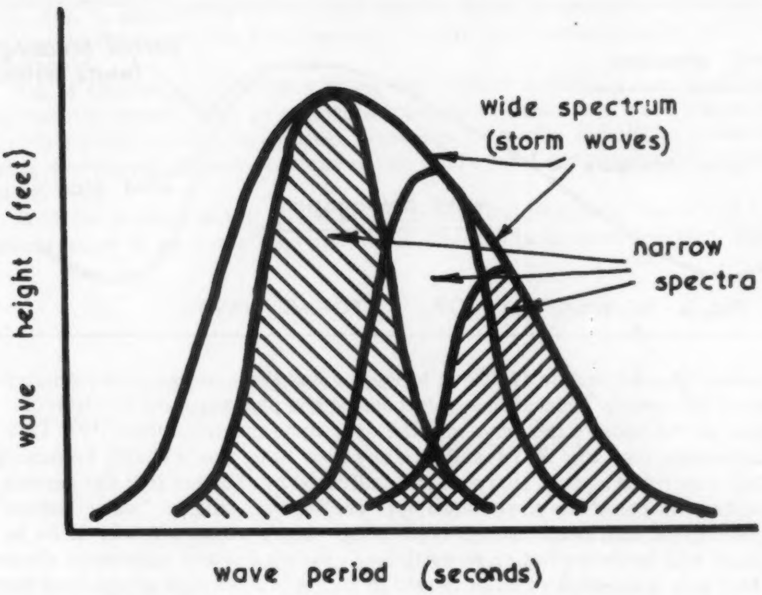

FIG. 1. SPECTRA OF OCEAN WAVES.

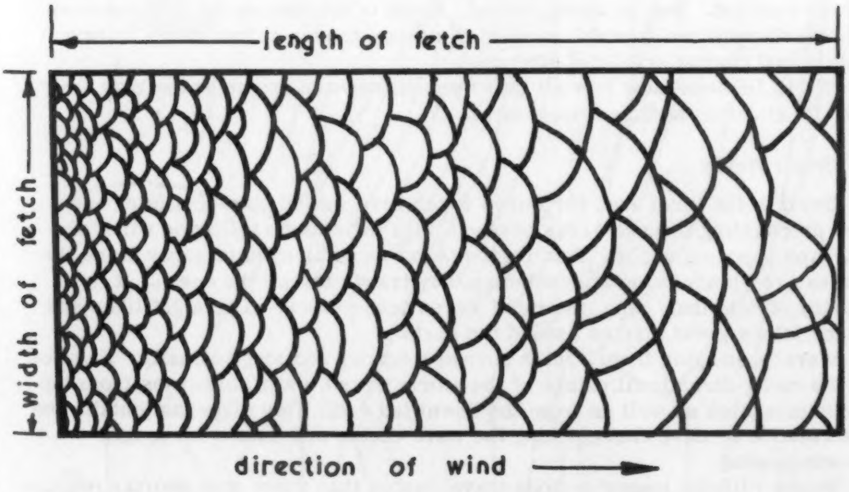

FIG. 2. WAVE CREST PATTERN WITHIN A FETCH.

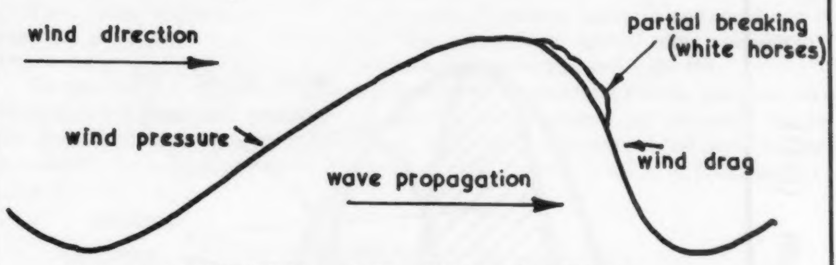

FIG. 3. ASYMMETRY OF STORM WAVES.

Another characteristic of storm waves is that their shape is asymmetrical about the crest. This is brought about by the pressure and friction of the wind on the back of the wave and the drag upon the front face.(14) This peculiar shape (see Fig. 3) causes storm waves to be more easily broken by shoaling water and contra currents, as indicated by the fact that the generation process is generally associated with partial breaking or "white horses".

As energy is still being transferred to the waves within a fetch, those in each train will increase in height until they reach a certain maximum steepness, that is a maximum ratio of height to length. When this is reached they break and probably this is the process whereby the longer wave lengths are created.(15) Although the maximum steepness for a single train of waves is $1/7$, the existence of many trains in a storm area means that this figure is never reached. But the asymmetrical shape of the waves, the interference of other waves and general transfer of energy causes storm waves to have this added characteristic of steepness.

It will be seen later how all these peculiarities of storm waves play their part in effecting sediment movement.

(b) Swell Waves

Swell is the term used for waves which have moved outside a fetch and are propagating through areas of ocean with little or no following wind, or perhaps opposing winds. This region is known as an area of decay since the waves are slowly being attenuated as they travel across the ocean. It is worthy of note that, once generated waves tend to move in straight lines and hence follow great circles around the earth.

Waves emerging from a fetch spread laterally and longitudinally. Because of the multi-directional nature of the storm waves, swell disperses from the sides of a fetch as well as from the downwind end. This dispersal entails the distribution of wave energy along the wave crests and makes for longer crested waves.

Waves with the longer periods travel faster than those with shorter periods and hence the various wave trains become spread over a vast area of ocean.(16) At a given point within the decay area waves will be experienced which initially are long with perhaps a single period. These will be followed by other wave trains of slightly differing periods which progressively become

smaller as the slower wave trains pass the recording point. This is depicted in Fig. 4, the narrow spectra are part of the total spectrum of the storm waves. In the figure the simultaneous conditions at several points are illustrated.

Fig. 4 also shows that these waves are mainly moving in one direction at any specific point. Only in a certain triangle of ocean will wave trains be arriving from several directions. Here again an over simplified situation is being discussed. In fact, a complex sea could exist over thousands of square miles.

With the sorting of the waves laterally and longitudinally there will be less interference of wave systems and hence smaller peak wave heights. Also the

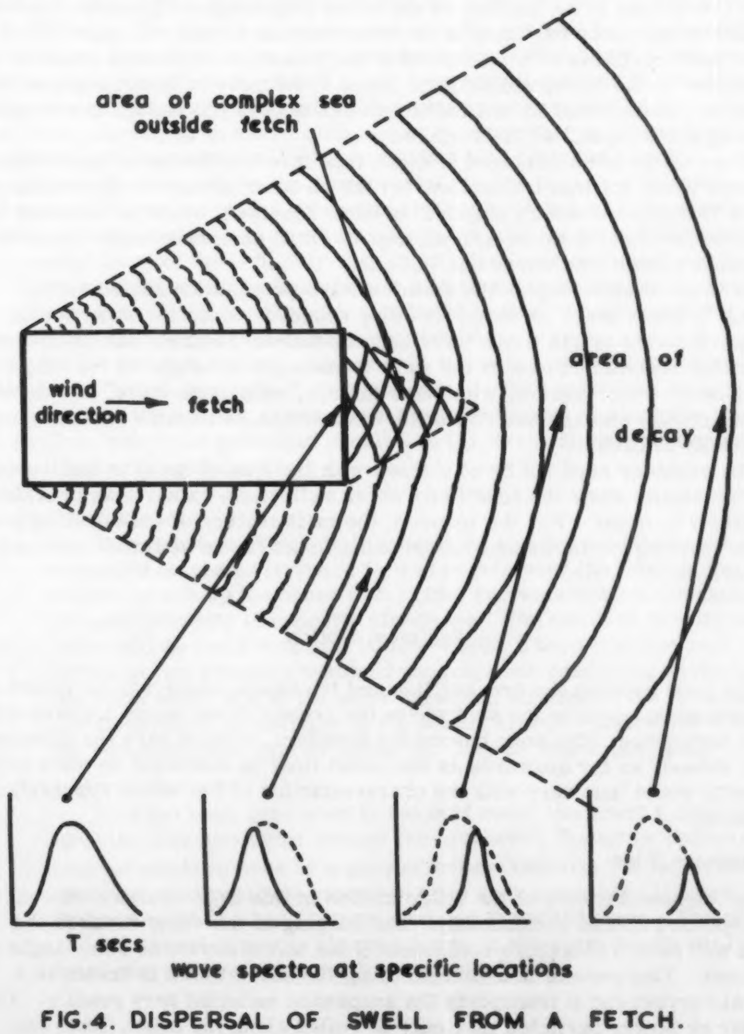

FIG. 4. DISPERSAL OF SWELL FROM A FETCH.

waves of any one train will be attenuated due to spreading[17] making again for smaller composite wave heights. Due to the narrow spectrum of the swell the interference of these trains causes maximum heights to occur infrequently between long periods of relatively calm water.

The above characteristics all have their effect in the motion of the sediment along the coast. Of course there are degrees of intensity between the extremes of swell and storm waves, but it is sufficient for this discussion to keep these effects separate.

Wave Motion

Wave motion in the surface of the ocean is propagated by an oscillatory motion of the water particles in an orbit which is almost circular.[18] The uppermost particles rotate in an orbit the diameter of which is equal to the amplitude of the wave, whilst those lower down move in diminishing orbits until, at a depth equal to half the length of the wave the motion is negligible, it being about 5% that at the surface.

When water is encountered of depth less than half the wave length (termed shallow water for that train of waves) the circular motion of the bottom particles is distorted and an elliptical orbit is assumed, which is reflected in particle movements throughout all depths. Still shallower water turns this elliptical motion into linear oscillations.

At these shallow depths the speed of wave propagation varies as the (depth) 1/2 and hence its length is being reduced. Since the wave energy (proportional to length times wave height squared) remains relatively constant this retardation causes the wave to increase in height as the length is decreased. This, together with other factors, causes the wave, at a depth approximating one and half times the wave height, to become unstable and eventually to break.[19]

The engineer need not be concerned with the type of breaker that results [20] but should study the conditions where reflection, rather than breaking, are likely to occur. For the moment, the main feature of the breaking process is that two distinct zones are identifiable as far as sediment motion is concerned.

Sediment Motion

The zone between the breaker line and the beach, which will be called the onshore zone, contains the surf where the broken waves surge forward with great turbulence. The zone beyond the breakers, termed here the offshore zone, extends as far seawards as the ocean floor is disturbed by wave action; its width would thus vary with the characteristics of the waves approaching the shore.

(a) Onshore Zone

The turbulent nature of the water motion in this area causes sediment to be suspended almost continuously. The surging of the water towards the beach will have a longshore component if the waves arrive at some angle to the coast. This results in a current along the shore which is known as a littoral current and it transports the suspended material very readily. The heavier or larger particles may only be rolled along the beach floor, but,

because of the angle of the uprush to the shoreline and the near normal return of the backwash, they are forced along the coast in a zig-zag fashion.

(b) Offshore Zone

In this area the water particles near the sea-bed oscillate backwards and forwards as the waves propagate overhead. Each oscillatory movement causes sediment to be sheared from the floor and when the motion is reversed a vortex is set up which throws the sediment into suspension. It quickly settles only to be picked up immediately by the next wave action.

The processes by which particles are initially set in motion have been discussed by Danel et al,[21] it may involve rolling, saltation, jumping and/or suspension. Bagnold[22] has illustrated that for a smooth bed the layer of water adjacent to it suffers a large lateral velocity gradient inducing turbulence, whereas above this layer there are quite calm conditions. Thus sand grains thrown up out of the turbulent layer have a good medium in which to complete their trajectory. Generally the particles are rotated by the action and are thus maintained in suspension longer than for normal translatory motion; Martinot-Lagarde et al[23] have studied this "Magnus effect".

Soon this motion results in the formation of ripples in the sedimentary bed. Bagnold[24] has discussed the parameters associated with the ripple formation, but the engineer need not be concerned with the actual dimensions of these undulations of the bed, only with their effect upon sediment movement. They induce greater turbulence and make for a wide band of turbulent water with longer mixing lengths. All this tends to keep the sediment in suspension for a longer period during each oscillation.

It is obvious that any current occurring in the water near the ocean bed will move this "quazi-suspended" material with it. The particles, in essence, will jump across the seabed, either from ripple to ripple, or along the crest of the ripples, which are generally parallel to the wave crests. Such currents may have the following origins:

(i) Tidal Range

This is not particularly important on an open coastline, but in estuaries or channels where flow is accelerated the tidal action can deepen or silt up the ocean bed. Tidal currents are not necessarily of equal intensity in opposite directions. The effect of density currents will be dealt with in a later section. There can also be a vortex action around a headland causing a current along sections of the coast which are in the same direction throughout the flood and ebb tides; such an example is the vortex around Portland Bill and the resulting tidal current along the Chessil Beach (England).

(ii) Littoral Current

This current, generated in the surf zone, can exert a drag force upon the adjacent water beyond the breakers. It may be deflected into the offshore area by a groyne or breakwater. The underwater contours soon protrude seawards near such structures illustrating that sediment has been transported and deposited there. The strong and infrequent currents almost normal to the coast fed by the littoral current are known as "rip currents".[25]

(iii) Wind Stress

When generating waves the wind exerts a strong drag upon the surface of the ocean which is felt at greater depths and a large body of water may be put in slow circulation. This may occur as a transient effect from a single cyclonic disturbance, or it may be a seasonal current due to repetitive meteorological conditions. Unless such currents are channelled through restricted passages to increase their velocities, they are unlikely to influence the movement of beach sediment greatly.

(iv) Water Elevation

As with tidal currents the effect of higher than normal water levels, caused by wind stress or low atmospheric pressures,[26] is felt more in shallow seas where steepening of the wave occurs (see section 3). This occurrence can cause flooding and break-throughs to inland waters and so drastically influence the movement of sediment along the coast.[27] Rip currents, as mentioned in (ii) above, can be caused by a series of high waves on one section of the beach which in essence raise the level of the sea surface temporarily at that point.

(v) Mass Transport

This is not a current in the same that a continuous forward movement of water is experienced. As previously stated the particle motion due to wave propagation is <u>almost</u> a circular orbit. In fact the particle does not quite complete the orbit and after the passage of each wave it is located a small distance in the direction of wave propagation. As a "current" therefore it would not be measurable with a current meter unless some integrating mechanism were used to compute the resultant of these continuous additons and subtractions.

This mass transport of water, as it is called, varies throughout the water depth and differs for the various wave characteristics. The engineer is mainly concerned with its strength and direction near the ocean bed where it is likely to affect the sediment movement. It has been shown experimentally[22] and mathematically[28] that a strong mass transport "velocity" occurs at the bed in the direction of wave propagation. It is strongest for small ratios of $\frac{\text{depth}}{\text{wave length}}$ (see Fig. 5). Hence, for a given depth, it is strongest for long period waves, and, for a specific wave train, increases as the depth decreases. The effect of several wave trains on this mass transport has not been analyzed. It seems that the resultant oscillations at the ocean floor would be similar to those for a small period wave train and hence mass transport would be minimized.

The illustrations cited above were for smooth bed conditions. Bagnold[22] found that when a sedimentary bed existed, with its consequent rippled surface, the forward motion near the bed was substantially reduced and that a larger band of water moved forward at a slower velocity. It could be inferred that the smaller the ripple amplitude the greater the mass transport.

Associated closely with the mass-transport effect is that of the differential forward and backward velocities of the water particles,

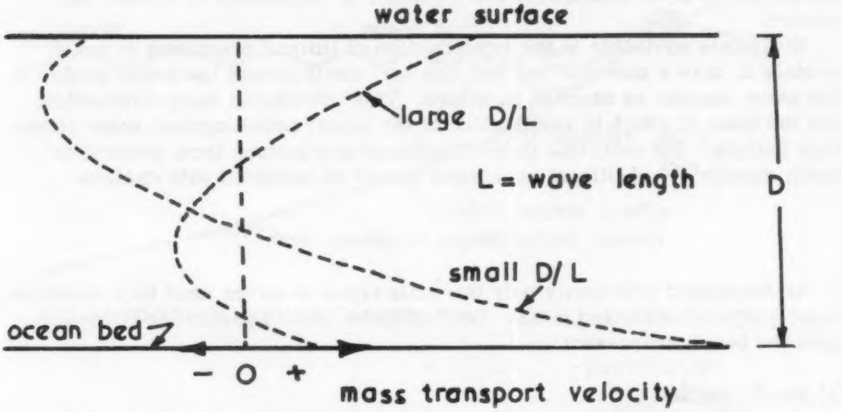

FIG.5. MASS TRANSPORT DUE TO PROGRESSIVE WAVES.

especially for waves in shallow water. It has been shown experimentally(29) that in such cases the forward velocity under a crest may be twice the backward velocity under a trough. This greater forward velocity gives momentum and rotation to the sediment which causes a net forward movement.(30)

(c) Complete Beach Profile

It is still believed by some authorities that the greater part of the longshore sediment transport occurs within the onshore, or surf zone.(31) When the bases upon which these convictions are held(32, 33) come to be analyzed it is found that the observations (mainly of scale models) were not comprehensive enough to vindicate them.

It is pertinent here to quote Bagnold,(22) in speaking of studies made of coastal sediment movement, he states:

> "But in a large number of practical problems of coast erosion and sanding up, the immediate inshore phenomena are controlled by the configuration and by slow movements of great sand accumulations which extend under shallow water far to seaward of the plunge line."

In comparing the relative volumes of material transported by suspension and rolling in the surf zone and by quazi-suspension, jumping and rolling beyond it, the apparent high velocities in the surf zone must be weighed against the very large area of seabed beyond it influenced by wave motion. Another factor (to be discussed later) is that the fine sediment is sorted offshore(34) and is more readily moved than the coarse material remaining near the beach.

There is an urgent need to develop an instrument for measuring the suspended and bed load movement both in the onshore and offshore regions. Watts has attempted to measure suspended load in the surf,(35) but, other than this, no success has been recorded in measuring these various rates of transport. The water movement, of course, is three dimensional and even

the presence of an instrument can influence it sufficiently to distort the record.

It appears advisable in the reproduction of littoral processes in scale models to have a movable bed that can sort itself across the beach profile in the same manner as happens in nature. This introduces many difficulties, not the least of which is verification of the model action against some phototype feature. But until this is accomplished conclusions from general or basic models(36) of littoral processes should be accepted with caution.

Beach Profiles

As mentioned previously only two main types of waves need be considered, namely storm waves and swell. Each of these has its particular influence upon the beach cross-section.

(a) Swell Profile

As the waves approach the coast, at some distance offshore, where the depth approximates half the length of the wave, sediment on the ocean floor is being disturbed. In the presence of several wave trains the longer waves are first to "touch" bottom. The mass-transport and its associated effects force the particles thus disturbed towards the shore. When the waves approach obliquely to the coast the sediment is moved along the shore as well as towards it.

As discussed in section (v) above, the process is amplified as the breaker zone is approached, but this is offset somewhat by the increasing slope of the ocean bed, resulting in sediment having to be swept "uphill". The depth and slope of the profile is modified until the shoreward tendency of the particles is balanced by the opposing forces and an equilibrium profile results.(37)

Even when this stage is reached a longshore movement could still proceed in the offshore zone due to the particles oscillating about a mean position which is continually shifting parallel to the beach due to the longshore components of the mass transport "velocity" and the differential speeds of the forward and backward oscillations as previously mentioned.

In practice such equilibrium as envisaged above is never reached. The wave heights and periods are continually changing, as also the wave direction. These new conditions bring about changes in size, spacing and orientation of the ripples. It has been found(38) that these occasions accelerate the sediment transport. It is readily understandable that, during the instability between the water and sand dune conditions, more turbulence exists and larger volumes of sediment will be moved in order to establish the new "regime".

The sediment, having reached the breaker zone, is carried forward with the uprush onto the beach face. Some of this water soaks through the beach sand, the remainder returning down the slope to the sea.(39) The volume and velocity of this backwash is substantially less than the uprush and hence much of the suspended sand is left stranded on the beach and so accretion occurs. A steeply graded beach results, as illustrated diagrammatically in Fig. 6.

With swell breaking on the beach there is a reasonable time between waves (especially the peak waves) for percolation to occur and for the above processes to take place. (See Fig. 7.) This presupposes that sediment continues to be supplied from offshore.

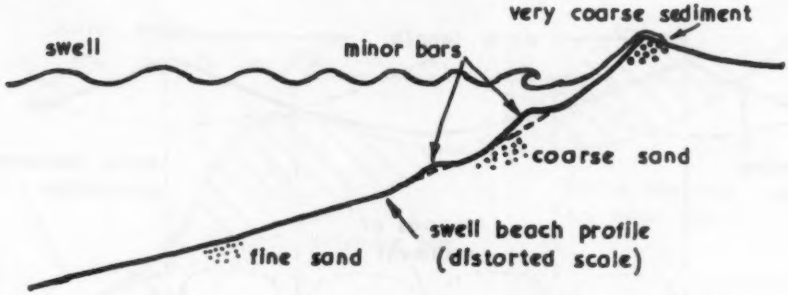

FIG. 6. PROFILE OF AN ACCRETING SWELL BUILT BEACH.

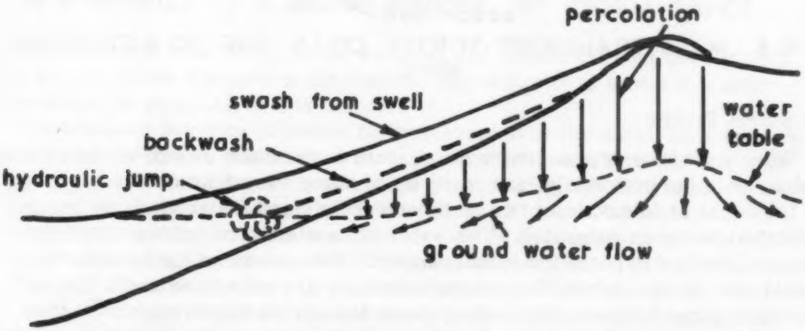

FIG. 7. HYDRAULIC CONDITIONS NEAR THE BEACH FACE DUE TO SWELL.

During this action the grains become sorted, the largest being thrown high on the beach, fine material being located in the surf zone and the finest appearing beyond it.(34) Each sediment has its own equilibrium slope for a given wave characteristic, the larger the grain size the steeper the slope.(40)

Pebble beaches stand at very steep grades which tend to reflect the waves rather than break them;(41) varying degrees of standing waves or clapotis are thus set up, which cause the oscillations of the water to assume such proportions that large boulders can be tossed about and so moved along the coast. The mass transport circulation in a standing wave has been shown to be(28) in vortex cells which build up sedimentary ripples at the quarter wave lengths (see Fig. 8.) This circulation is accompanied by high velocities and great turbulence and hence increased volumes of material are thrown into suspension.

Steep beaches are likely to cause only partial standing waves. Near-vertical walls, on the other hand, can induce almost complete clapotis formation. But under any of these circumstances, a long-shore current is assisted greatly in its transport of sediment along the coast. This action, by rocky headlands, of expediting the movement of sand has only recently been appreciated;(42) it is generally hidden from view although Hamada records having observed it.(43)

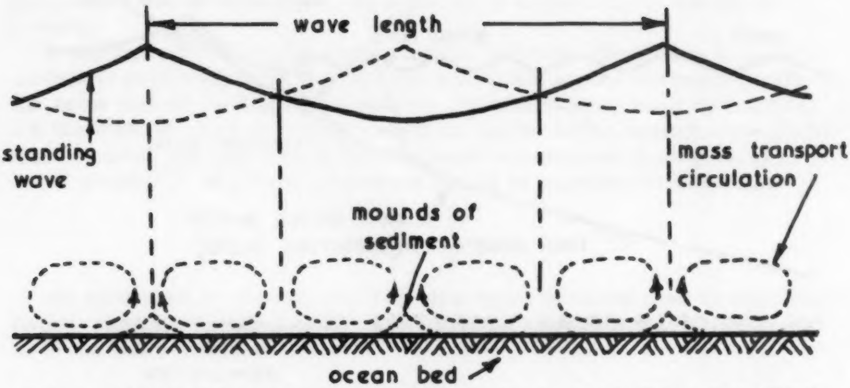

FIG. 8. MASS TRANSPORT VORTEX CELLS DUE TO A STANDING WAVE.

(b) Storm Profile

When a fetch or storm centre is adjacent to the coast storm waves arrive which, as noted previously, are made up of many wave trains. Waves break on the beach at intervals of two or three seconds and the sand of the beach face soon becomes saturated. The water table almost coincides with this face and further percolation is impossible. The volume of the backwash equals that of the uprush, the return velocities are increased and a larger hydraulic jump forms as this water meets the succeeding trough.[39] (See Fig. 10).

Great erosive powers thus exist added to which the sediment near the base of the beach face is in near-suspension due to the ground-water flow through the face back to the sea.[44] This is easy prey to the swirling backwash. It is no wonder that beaches can vanish in a matter of hours after the onset of a storm.

The water, which is literally poured onto the beach, must return to the sea, being laden with sediment it does so across the sea floor. When, at some depth, its velocity is sufficiently reduced, deposition occurs and a sand bar is built some distance offshore. When this is sufficiently large succeed-

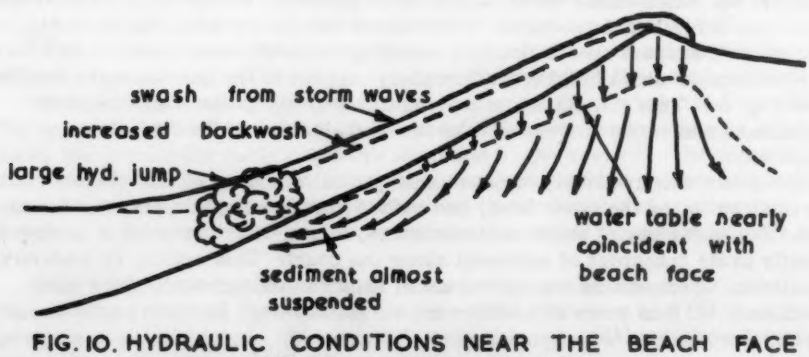

FIG. 10. HYDRAULIC CONDITIONS NEAR THE BEACH FACE DUE TO STORM WAVES.

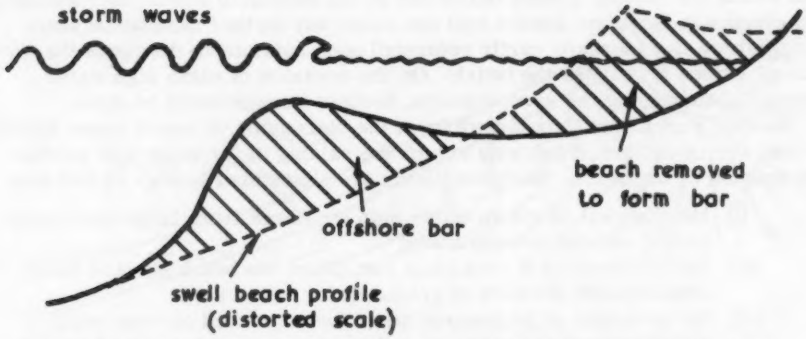

FIG. 9. PROFILE OF A BEACH ERODED BY STORM WAVES.

ing waves break over it due to the shoaling and contra current over it (see Fig. 9). Previous discussion elicited the fact that storm waves are more susceptable to breaking in this fashion.

The offshore bar thus prevents further erosion of the shore because only broken and dissipated waves arrive at the receded beach line. In this way the complete destruction of coastlines is prevented by the sedimentary material being oriented into a form of defence line. It can be inferred that to safeguard structures located near the beach it is necessary to have a reserve of sediment to support them after allowance has been made for a certain loss of beach to the offshore bar. It will be shown later that the amount required for such a bar can vary with time depending upon the general longitudinal stability of the coastline.

The multi-directional nature of the storm waves (see section 2a) discourages the formation of a strong longshore current (except where lateral feeders supply water to a rip current). The general result, therefore, is that the material of the bar is moved directly offshore from the beach. Two other factors which reduce the effect of any longshore transport during a storm sequence are (i) the fetch changes direction very quickly as the cyclonic centre moves across the coastline (to be discussed more fully later), and (ii) the duration of storm conditions on the coast throughout the year is insignificant when compared with the duration time of swell.

(c) Beach Cycles

The swell waves which subsequently arrive at the shore following a storm, generally from an oblique direction, return the bar material to the beach up- or down-coast from its original position. The storm waves thus play a significant part in assisting the swell to move sediment along the coast. Should there be a predominant swell from one direction, which is the case in many parts of the world, then there is a net movement of sediment resulting from it.

An inference to be drawn from the above is that model reproductions of littoral drift processes should allow for the annual sequence of storm and swell waves in duration and frequency. This presupposes an ability to generate representative storm conditions in the model.

From section (b) above it is apparent that the swift and alarming erosion of beaches is brought about by the first storms of the season, whose waves

can attack the steeply graded beach left by the season of swell. Any succeeding storms in the same season will not cause any further denudation since the offshore bar (perhaps partly returned) soon reforms to dissipate the wave energy before it reaches the beach. On the occasion of extra high water levels accompanying the second storm, further damage could be done.

Another conclusion to be drawn from the discussion on beach water tables is that any conditions which may induce the raising of the table will accelerate erosion of the beach. Such conditions are illustrated in Fig. 11 and are:

(i) the disposal of storm water onto the beach from large catchment areas, natural or man made;
(ii) the existence of a rock shelf just below the beach surface which concentrates the flow of ground water to the sea;
(iii) the presence of promenade walls (usually sited on base rock) which prevent the temporary flow of ground water inland when the storm sea level exceeds the normal water table level.[44]

Even so, the existence or non-existence of the above features will not greatly affect the disposal of the hundreds of tons of water which are thrown onto the beach during the course of an hour's storm. Only the provision of sufficient width of sedimentary material for the formation of a bar and for a surplus can provide the ultimate defence against erosion of such coastlines.

Longitudinal Equilibrium

The equilibrium shape of a coast in plan is determined, firstly, by the fixed points on the coast such as rocky headlands or man-made structures of the groyne or breakwater type, and, secondly, by the predominant direction of the sediment movement along the coast. The latter will be considered first.

(a) Meteorological Conditions

There are three types of fetch which may assume importance in littoral drift and its associated processes. They are:

(i) cyclonic or low pressure centres
(ii) anticyclonic or high pressure centres
(iii) sea-breezes.

Of (i) and (ii) the more important is the cyclone because the winds accompanying it are the stronger by far. Anti-cyclones may occur more frequently and for longer periods, but their wind velocities are not great.

Sea-breezes can be quite persistent and can have speeds and fetch lengths sufficient to generate steep, short period waves, which do not break until right on the beach. They suffer little refraction and hence break at sharp angles to shore, thus generating a strong littoral current in a narrow band of water. The actual influence of the sea breeze in the littoral drift process is hard to assess because of a dearth of information regarding wind velocities and duration, as well as fetch lengths.

Hence, the study of coastal equilibrium should be based upon data regarding the location, duration and intensity of the low pressure or cyclonic centres in the adjoining seas. This can provide certain qualitative characteristics of the waves arriving at given points on the coast, including the direction of ap-

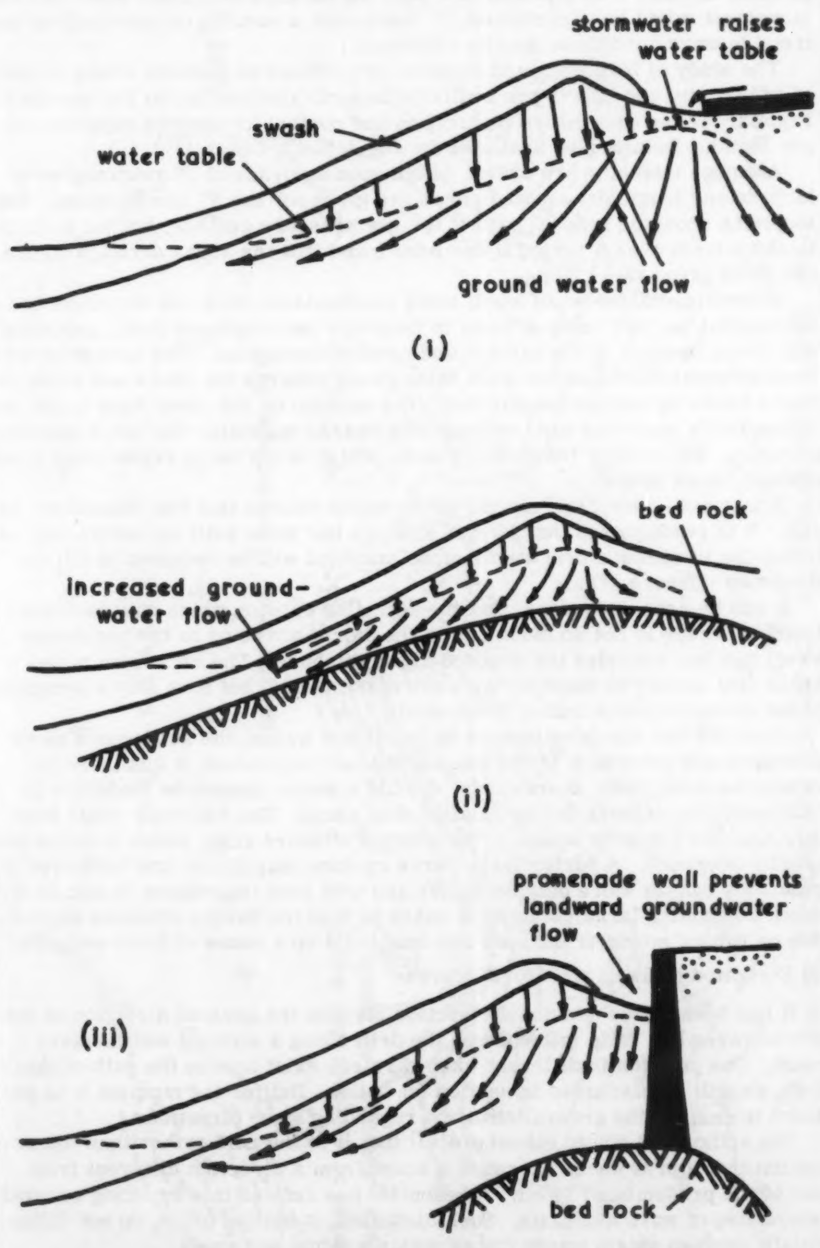

FIG. II. CONDITIONS CONDUCIVE TO BEACH EROSION.

proach. Studied over a period of a year the balance of upcoast and downcoast movement might be ascertained. If taken over a number of years a long term trend in wave conditions may be exhibited.

The study of frequency and duration of cyclones in specific areas is that of cyclonicity and charts are available in Australia similar to the sample in Fig. 12. These show hours of duration and number of centres experienced per month. Data is also available for the Atlantic Ocean.(45)

Although isopleths are drawn, based upon analyses of 1° quadrangles of latitude and longitude, actual figures are given for the 5° quadrangles. The isopleths show the general path of the low pressure centres, but for a statistical analysis over a period it has been found that the hours duration figure can be of great value.(46)

In the circumstances of swell being predominant from one direction and a substantial barrier being present to interrupt the longshore drift, accretion will occur upcoast of the barrier and erosion downcoast. The accretion will be seen immediately as the sand being swept towards the shore and along the shore banks up against the barrier. The erosion on the other hand is not felt immediately since the sand coming shorewards maintains the beach line temporarily. But in time the offshore area, which is not being replenished from upcoast, must deepen.

It may not be until the arrival of the storm season that this denudation is felt. It is readily seen that for the offshore bar to be built up sufficiently to break the incoming storm waves more material will be required to fill the deepened offshore area.

It can be seen, therefore, that the alarming erosion which causes coastlines to recede is not so much due to the storm waves as to the persistent swell that has provided the denuded areas offshore. The recession of the beach line occurs in steps on such storm occasions, but it is only a symptom of the more insidious action of the swell.

After the bar has been formed by the storm waves, the subsequent swell acts upon and returns it to the beach. This accomplished, it continues to sweep the ocean floor shorewards. Should a storm season be moderate in character the offshore bar is smaller than usual. The following swell therefore has more time to attend to the general offshore area, which is more than usually deepened. A particularly fierce cyclone may follow and the waves it generates can, at more oblique angles and with less impedance, break on the beach demanding large volumes of beach to feed the hungry offshore depths. One or two calm winter seasons can thus build up a sense of false security.

(b) Direction of Swell and Storm Waves

It has been shown previously (section 5b) that the general direction of the storm waves has little influence on the drift along a straight sedimentary coast. The position is different when barriers exist across the path of this drift, as will be discussed in section (c) below. But for the moment it is pertinent to analyse the general situation regarding wave directions.

The author has stated elsewhere(47) that it is the rule rather than the exception that storm waves approach a coast from a direction different from that of the predominant swell. Johnson(48) has refuted this by citing several references of wave hindcasts. Such hindcasts, it will be found, do not differentiate between storm waves (those within a fetch) and swell.

Johnson also cited an article by Spring(49) on waves encountered at Madras Harbour. Spring wrote of this: "The former class of waves (S.W.), continu-

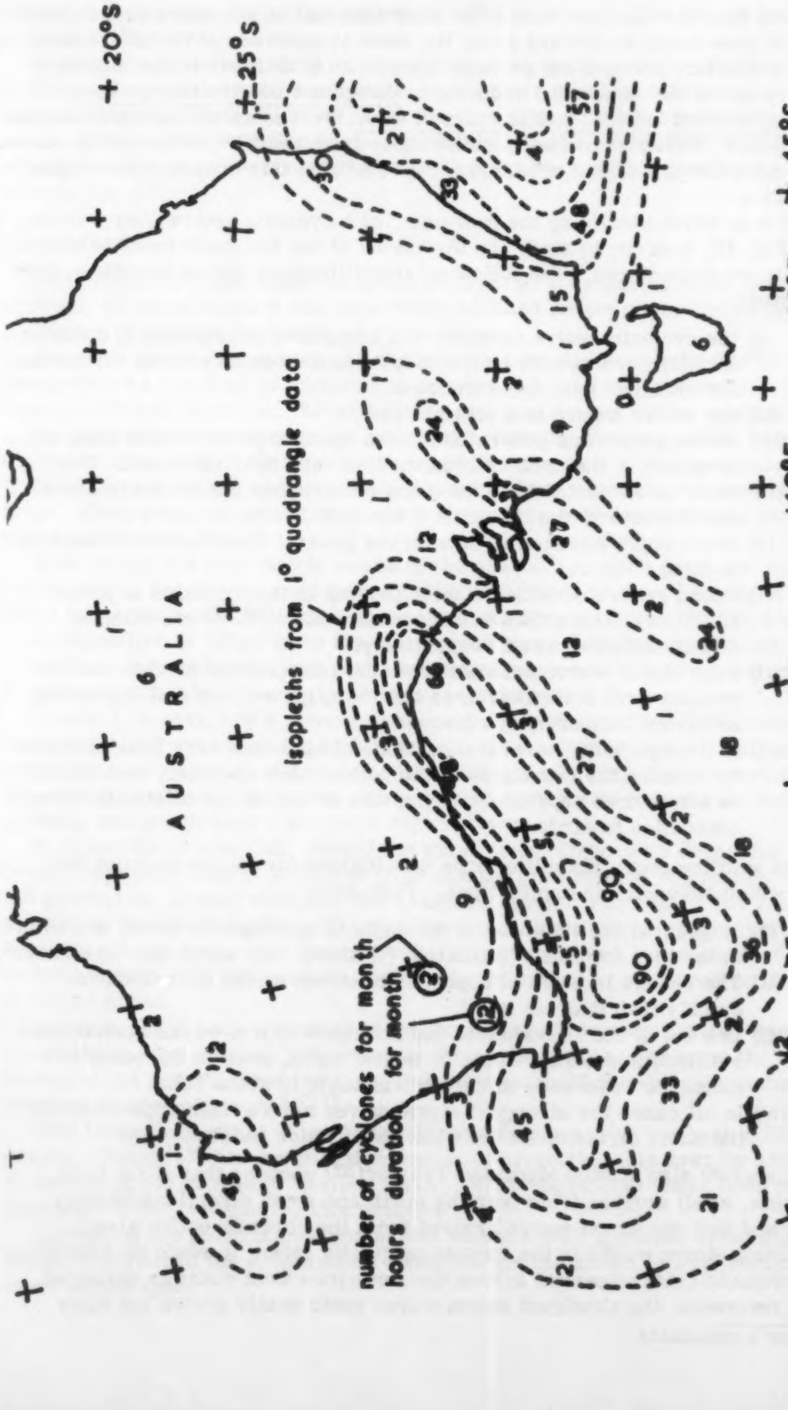

FIG. 12. CYCLONICITY CHART OF AUSTRALIA FOR JUNE 1951.

ing from March to August, keeps the sand near and on the shore in a constant state of slow creep northward along the several hundreds of miles of coast, while the latter, acting from perhaps the middle of October to the middle of January shifts the same sand southward, though not so effectively—near Madras, at least—as the former class of wave.** The author infers from this that the S.W. waves were swell, whilst those from the N.W. were storm waves, based upon the previous discussion on wave action; this supports his original contention.

But it is worth idealizing the movement of a cyclonic centre near a coast, as in Fig. 13, in order to study the directions of the two main types of wave likely to be encountered. The following simplifications and assumptions have been made:

(i) the cyclonic centre consists of a circular wind structure, directed slightly centrewards and with a definite boundary (wind circulation for northern hemisphere shown).
(ii) the centre moves in a straight line
(iii) storm waves are generated for the specific points shown when the boundary of the cyclone is across or very near the coast.
(iv) swell is encountered at the same points when the centre is about one diameter from the coast (or more)
(v) swell which has spread outside the general direction of its fetch is omitted
(vi) waves generated with the fetch moving in the direction of propagation are larger than in the contrary case[50,51] resulting in strong and weak swell respectively.
(vii) weak storm waves emanate from the fetch direction when the cyclone first enters the area and these grow progressively stronger as the cyclone advances through it.
(viii) in the figure the several directions of approach have been included by drawing the coastlines in the paths of the cyclones, each should be considered separately so that the others do not affect the wave generation towards it.

Even with the above simplifications conclusions can be drawn from the figure which have certain applications. They are:

(i) In general the variation in the angle of approach of storm waves is wider than for swell (limitation (v) above may affect this conclusion).
(ii) The change in angle of approach of storm waves occurs more quickly than that for swell.
(iii) In 9 out of the 11 instances the strongest storm waves arrive from a different quadrant to the strongest swell, even in the other two cases the difference in the arrival angle is about 70°.
(iv) In all cases the strongest storm waves arrive from approximately the same direction as the weak swell which follows.

Johnson[48] also quoted Munk and Traylor[52] showing that at La Jolla, California, swell arrives from both the north and south directions (mainly north), and that the storm waves arrived from the north direction also. Maintaining storm waves in the context originally given, it would be seen that, if the cyclonic centres passed across the American west coast as indicated in this reference, the strongest storm waves could easily arrive the other

* Author's emphasis

side of the normal to the strongest swell. The particular example of La Jolla is noted in Fig. 13.

The author submits, therefore, that in general storm waves, whilst approaching from widely different angles, have their greatest influence on the coast at angles greatly different from the strongest or most predominant swell. Such a generalisation is not "misleading to the novice"(48) but is a situation which must be accepted and applied in the design of maritime structures.

(c) Groynes

The groyne has been used for many decades as a defence against beach erosion. In some cases it has apparently fulfilled its purpose, in others it has failed to prevent recession of the coastline.

Consider the consequences of constructing a groyne perpendicular to the beach where a net drift of material is occurring due to the predominant swell from an oblique direction. Immediately accretion occurs on the upcoast side of the structure from whence the sediment is coming (see Fig. 14a), whilst the water line there moves out to the tip of the groyne the downcast side suffers erosion by not having replenishment for the material removed therefrom. Even when material can by-pass the groyne this denudation is never quite made good.

Now should the first storm waves of the season approach from the downcoast side of the groyne a rip current will be induced as shown in Fig. 14b and a large volume of material transported out deep into the offshore zone. This deposition is offset from the coast much more than the normal bar formation, and the subsequent swell can therefore move it further downcoast in bringing it ashore than if the groyne had not existed.

Hence it is seen how a groyne, built ostensibly to impede the movement of sediment along the coast can in fact accelerate it. This action is multiplied in the case of a system of groynes. As illustrated in Fig. 15 the coastline assumes a saw-tooth shape and storm waves, either side but near to the normal, can precipitate a series of rip currents.

It is worthy of note that, should the whole area offshore be deepening due to non-replenishment of sediment from upcoast, the system of groynes cannot prevent it. It may stabilize the coastline temporarily, but at the risk of distorting the beach profile. The risk is that a future fierce storm, perhaps accompanied by extra high water levels, may tear away a large width of beach, making breaches in weak coastal defences, and result in catastrophic flooding of inland areas.

An example of such a menace is the groyne system on the Danish west coast where "the groynes had, to some extent, been able to prevent the receding of the coastline but the large depths down to 20 metres approached the coast at an alarming rate".(27)

The immediate effect of such rip currents is damage to the heel of the groyne. Bruun(53) has investigated means of protecting this area by adding to the toe of the groyne in various ways. Other suggestions have been put forward(54) for overcoming this problem.

Accepting the general conclusion that storm waves approach from a different quadrant to the predominant swell the groyne shown in Fig. 16 has some advantages.

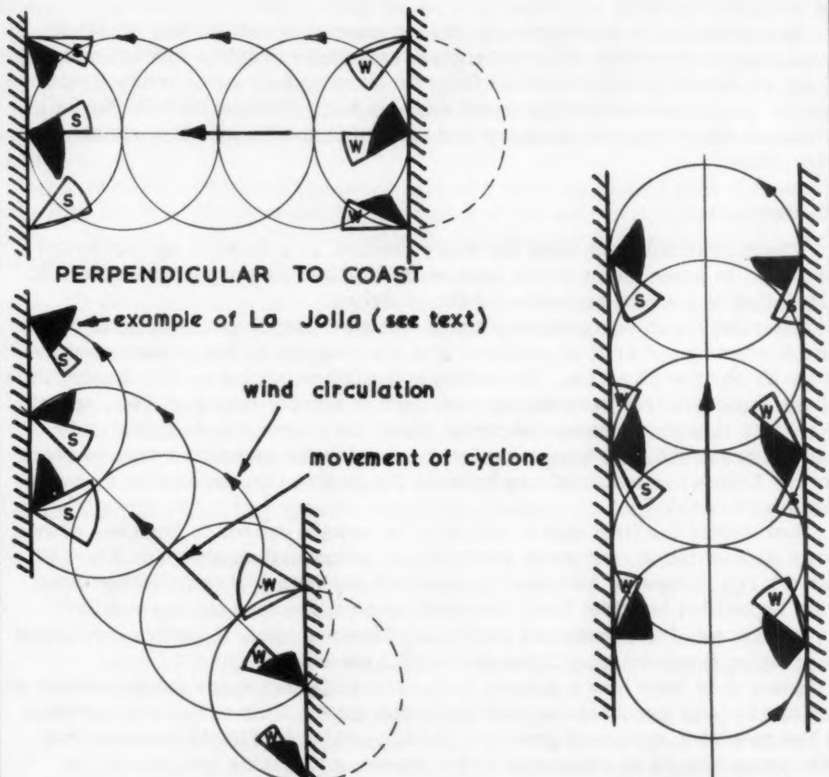

FIG. 13. WAVE OCCURRENCE DUE TO IDEALISED CYCLONIC CONDITIONS.

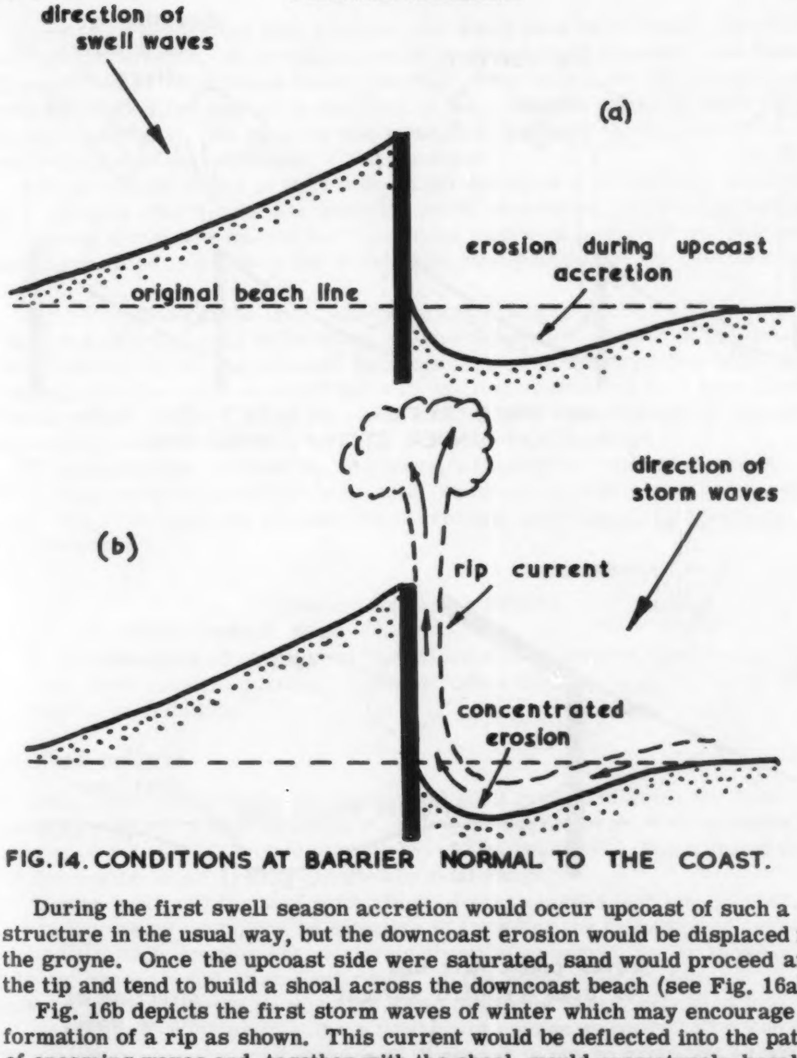

FIG. 14. CONDITIONS AT BARRIER NORMAL TO THE COAST.

During the first swell season accretion would occur upcoast of such a structure in the usual way, but the downcoast erosion would be displaced from the groyne. Once the upcoast side were saturated, sand would proceed around the tip and tend to build a shoal across the downcoast beach (see Fig. 16a).

Fig. 16b depicts the first storm waves of winter which may encourage the formation of a rip as shown. This current would be deflected into the path of oncoming waves and, together with the shoal, would prematurely break them. The current may even by deflected into a vortex. The likely consequence is that a large body of sand will be retained on the downcoast side of the groyne, protecting it, and providing a supply of sand to mitigate the next season's erosion.

But it must be remembered that groyne systems cannot prevent the recession of the coastline if the supply of sediment upcoast is insufficient to maintain the equilibrium profile of the beach. The best safeguard against catastrophe is a hydrographic survey of the offshore area every two or three years to see that the offshore bed contours are maintaining their position.

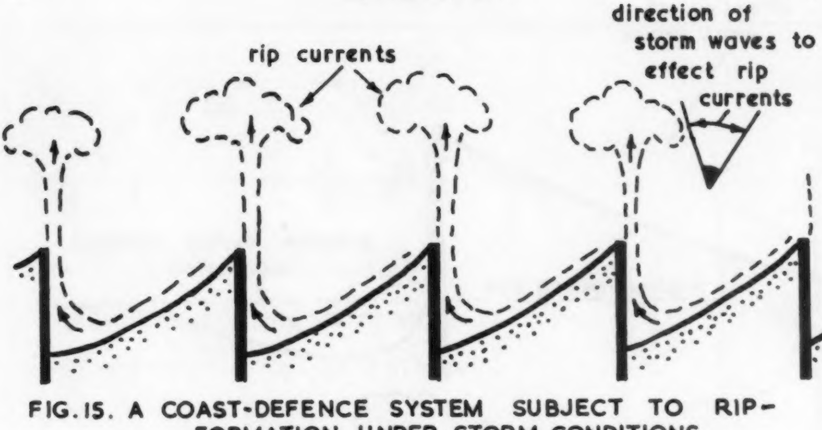

FIG. 15. A COAST-DEFENCE SYSTEM SUBJECT TO RIP-FORMATION UNDER STORM CONDITIONS.

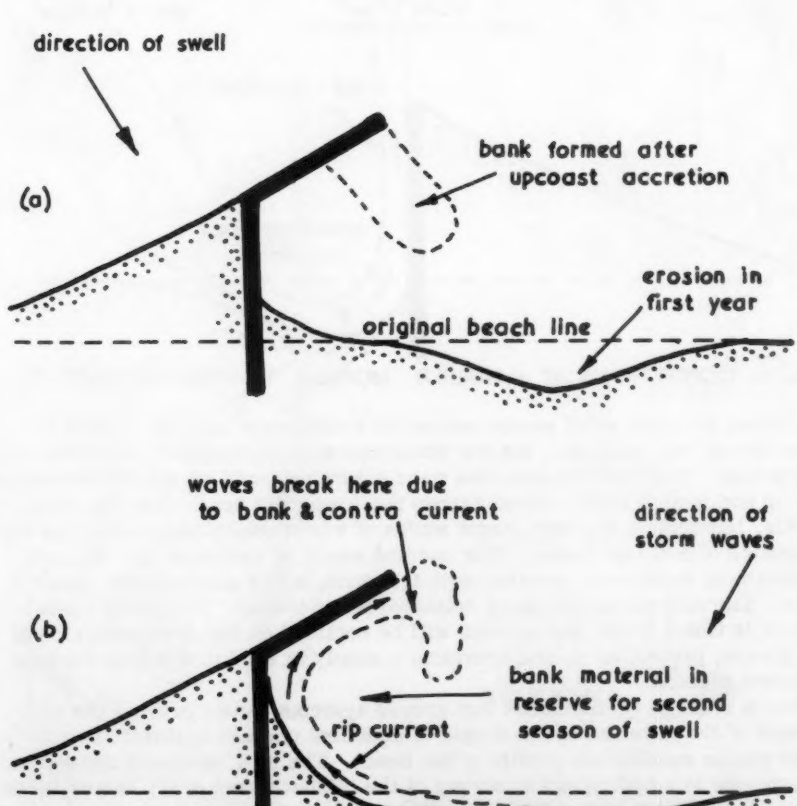

FIG. 16. BARRIER TO MINIMISE DOWNCOAST EROSION.

Sometimes associated with groynes, sea walls have been used to defend land against erosion. A recent innovation in this type of structure has been the asphalted or impervious beach face.[55] This runs from the lowest water mark up beyond the highest to the limit of the maximum expected wave run-up (approximately). Its purpose of course is to prevent the erosion of the beach and hence the recession of the coastline.

The immediate effect is to inhibit the formation of a storm-built offshore bar. This, in effect, puts this material out of circulation, but from previous discussion it will be realized that longshore motion of sediment can still take place, expedited perhaps by the wave-reflecting qualities of the new beach slope.

Since recession of the coast has instigated the construction of the "sea wall", the offshore area is likely to become deeper and deeper as the sediment removed is not replenished from upcoast. The beach profile will thus steepen until the coast on which the wall is cited is standing on a very dangerous slope. Subsidence of the whole coastal area is then likely on the occasion of a fierce storm.

This pessimistic attitude is, of course, a long term view and no doubt temporary relief is available from such construction, which may be economical. But here again the advisability of frequent hydrographic surveys could be reiterated.*

Tidal and Estuarine Effects

So far the effect of water level variations and the role of specific currents has only been noted in passing. Their influence on wave propagation is worth further consideration.

(a) Tidal Range

On an open coast where the continental shelf rises uniformly the tidal range results, in a raising and lowering of the water level. This spreads the energy of the surf zone over a greater width of beach.[56] The width of the offshore zone is not greatly affected by tidal range.

The spreading of the surf action in tidal areas means that low groynes, run out to low water mark, can be effective in impeding the transport of sediment.[57] Such groynes are relatively cheap to construct, unlike those on a steeply graded beach of no tidal range. In the latter case economic considerations prevent an effective width of profile being controlled.

The flatter beach profile of the tidal coast causes waves to be broken further offshore and the swift erosion of the beach is not so prevalent as with the almost tideless stretches of coast. Many articles could be cited to illustrate the fact that coasts most affected by erosion have tidal ranges up to five feet approximately.

The effect of tidal streams on waves has been discussed by Unna[58] and proved by experiment.[59] A stream opposite in direction to the propagation of the waves tends to stop them and break them. A flood tide, on the other hand, running with the waves carries newly arrived waves into those already in the slack water near the shore. This addition of the wave trains makes for

* The question of stable shapes of coastline in plan warrants a separate discussion.

a wide spectrum of waves and quasi-storm conditions. This tendency for erosion is offset by the spreading of the breaker energy over a wider area as the water level starts to rise, which also provides the beneficial effect of a lagging water table (see section 5).

(b) Estuaries

An estuary is defined as an area of sea which is affected by the discharge of fresh water into it. Stratification occurs due to the differing densities; the fresh water overlays the salt water which penetrates the estuary in the form of a wedge.

The immediate consequence of this wedge formation is that the water moving into the estuary on the flood tide is concentrated near the ocean bed and thus moves sediment upstream. During the ebb-tide the water issuing from the estuary is mainly near the surface. In fact, on the change of tide, water may be travelling in opposite directions at the surface and near the bed. This has been illustrated many times in the past and its influence on siltation is well known.[60]

Now the effect of tidal streams on waves (noted in the previous section) is to encourage this siltation process. The flood-tide carries waves into the estuary, making them longer and so helping them to "reach" the bottom and churn up the sediment of the bed. In spreading their energy over a longer wave length, they flatten, are less likely to break, and so penetrate far into the estuary (see Fig. 17). The ebb-tide, on the other hand, with its surface outflow, tends to shorten the waves and break them and so prevent them penetrating the estuary or disturbing the bed material.

The waves which are refracted to the sides of the estuary are carried in by the flood tide and broken at sharp angles to the shore. This generates a strong littoral drift into the estuary. In fact the overall tendency is for estuaries to silt up.

CONCLUSIONS

From the foregoing discussions it may be concluded that

(a) There are mainly two types of wave important to consider in the littoral drift process, namely: storm waves of wide spectrum, and swell of narrow spectrum.
(b) It is probable that the larger portion of the longshore transport of sediment occurs outside the breaker zone and not within it.
(c) Two typical beach profiles result from the wave systems mentioned in (a) above: the storm profile with its offshore bar, and the swell profile which is steep and usually accreting.
(d) In the storm profile, once the bar is of sufficient size to precipitate the breaking of oncoming waves no further erosion takes place; thus the first storms of winter cause the most denudation.
(e) Storm waves are much less frequent and of shorter duration than the swell, which therefore determines the net littoral drift and hence the physiography of the coastline.
(f) Storm waves are only part of the cycle of beach erosion, being instrumental in placing material offshore for the swell to act upon; the original cause of the erosion is the predominant swell associated with non-replenishment of sediment in the offshore area.

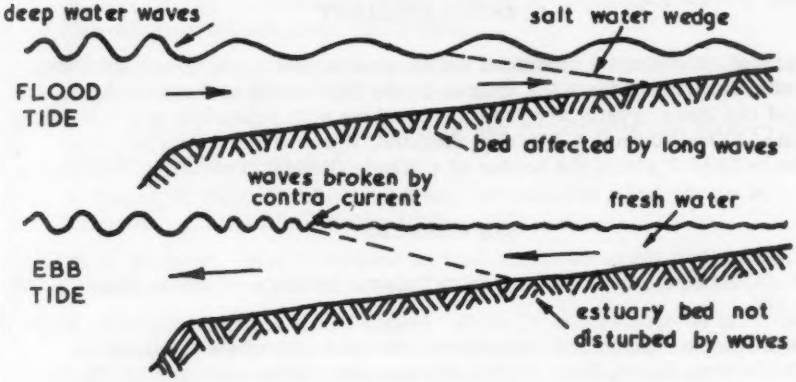

FIG. 17. EFFECT OF ESTUARINE CONDITIONS ON OCEAN WAVES.

(g) Any natural or man-made structures that restrict the percolation of uprush water through the beach-face accelerate the erosion of it.
(h) Cyclones provide the major fetches for waves influential in the littoral drift processes.
(i) Cyclonicity charts can be used for determing suitable years of wave hindcasting or for comparing survey periods of littoral movement with average conditions.
(j) Storm waves tend to arrive at widely varying angles to the coast and, in general, in a different quadrant from the predominant swell.
(k) Groynes constructed normal to the coast can accelerate coastal sediment movement by sponsoring rip formation.
(l) A groyne shaped as a half Y can maintain a coastline better than the type mentioned in (k).
(m) A good insurance premium against catastrophic losses in a "groyne supported" receding coastline is a periodic hydrographic survey of the offshore zone.
(n) The larger the tidal range on a coast the less likely are storm waves to denude the beach "overnight".
(o) Wave and current conditions tend always to silt-up estuaries.
(p) Littoral drift models should be able to reproduce to some scale:
   (i) the cycle of waves in direction and duration time
   (ii) the types of waves in their correct sequence - storm and swell
   (iii) the sorting of the movable bed material across the whole beach profile
   (iv) the same rate of sediment replenishment, to the area under study, as in the prototype
   (v) the rolling and jumping of grains and the consequent ripple formation
   (vi) the breaking or reflection of the waves
   (vii) the currents in the prototype - tidal, littoral or meteorological
   (viii) the percolation of uprush water through the beach face

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WATERWAYS AND HARBORS DIVISION

Proceedings of the American Society of Civil Engineers

PRESENT STATE OF COASTAL ENGINEERING IN JAPAN

Kiyoshi Horikawa[1]

ABSTRACT

Japanese engineers are striving successfully to improve the status of coastal engineering in Japan. A complete bibliography of Japanese publications on this subject is presented.

INTRODUCTION

As is well known, Japan is formed of small islands surrounded by the Pacific Ocean and Sea of Japan. The coast line is long compared with the area; that is, the coast line is 16,214 miles and the area is 142,338 square miles (Fig. 1). The interior geography is primarily mountainous area, and so the narrow band of open fields along the coast plays an important role in the economic and cultural activities of Japan. Most of the large cities have developed in the coastal regions and have become hubs of the national economy. One of the foremost problems in Japan is that of recurring damage in the populated coastal areas caused by tidal waves and extremely high tides. It is the goal of coastal engineers to protect coastal areas from damage due to natural forces and thus preserve these areas for optimum economic development.

Since the end of World War II, Japan has undertaken a large program of reconstruction of its public facilities consisting primarily of coastal engineering obligations, such as harbors and other coastal structures. This program of reconstruction can be classified into the four groups discussed in the following paragraphs.

Maintenance of Estuary Harbors

Ancient trade routes and trading centers developed what are now the principal economic centers of modern Japan. A problem exists in this country

Note: Discussion open until February 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2169 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. WW 3, September, 1959.

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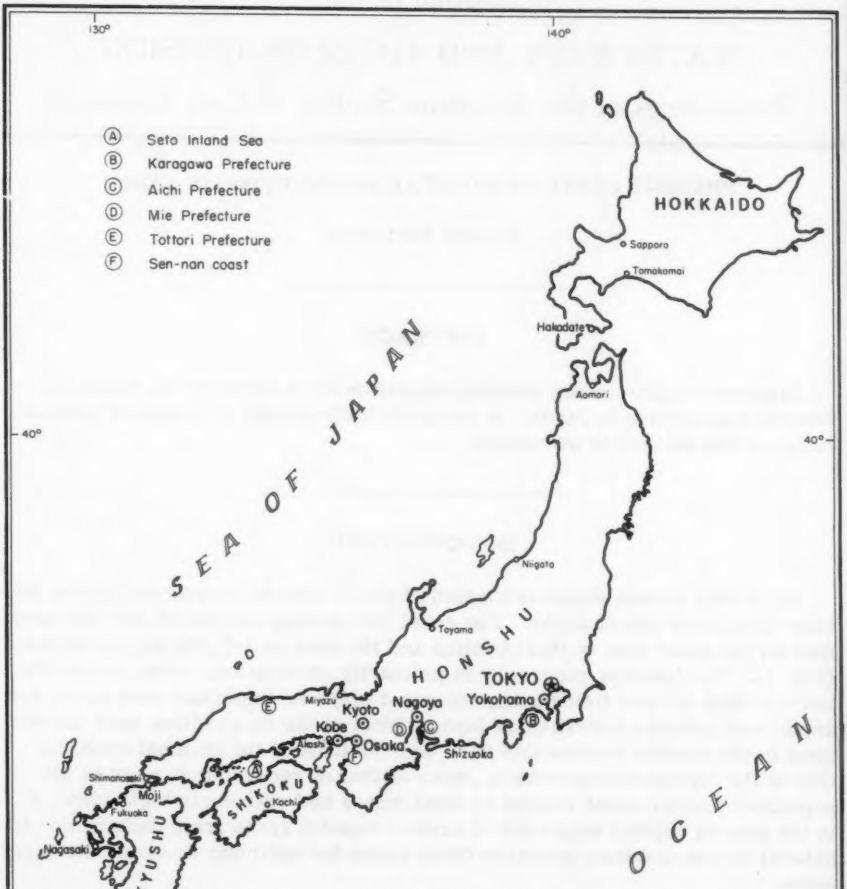
130°
140°
40°
40°
130°
140°
Ⓐ Seto Inland Sea
Ⓑ Karagawa Prefecture
Ⓒ Aichi Prefecture
Ⓓ Mie Prefecture
Ⓔ Tottori Prefecture
Ⓕ Sen-nan coast
HOKKAIDO
Sapporo
Tomakomai
Hakodate
Aomori
SEA OF JAPAN
Niigata
Toyama
HONSHU
TOKYO
Yokohama
Miyazu
Kyoto
Nagoya
Kobe
Akashi
Osaka
Shizuoka
Shimonoseki
Moji
Fukuoka
SHIKOKU
Kochi
Nagasaki
KYUSHU
Miyazaki
Kagoshima
PACIFIC OCEAN

FIGURE 1

wherein modern ships are being forced to use the antiquated facilites of existing harbors and are being barred from many ports by shallow channels. Navigable channels that were adequate for light-draft sailing ships of the past are not adequate for the large tankers of today. Most of the harbors in Japan, except Yokohama, Kobe, Nagasaki, etc., are located in the estuaries of small rivers; Tokyo, Osaka, Nagoya, and Niigata are typical examples.

An estuary, the junction between salt water and fresh water, is the place where the sediment transported by various currents has deposited heavily. To overcome this difficulty, dredging has become indispensable in maintaining channel depths necessary for the operation of ships of large draft. When a waterway, such as an estuary, is deepened artificially, the channel has a tendency to restore itself to a natural condition; this is done by heavy sediment depositon. Such deposition in the mouths of navigation channels can be reduced by the construction of jetties which constrict the flow, increase the velocity, and thus keep sediments in suspension. The planning and design of these structures are quite difficult, as they have conflicting requirements; that is, 1) the mouth of the river should be opened sufficiently to permit rapid flood discharge without the severe inundation due to the backwater effects, and 2) it should be kept sufficiently narrow to maintain navigable depths. Such coastal structures often cause severe beach erosion along the downdrift shore. A general tendency in modern practice is to separate the harbor areas from river areas by artificial works.

Prevention of Blocking of River Mouths

The description above is but one aspect of estuary treatment in water utilization. Another common problem in river improvement is the natural blocking of river mouths. This phenomenon is very complicated, because of the interaction of river flow, waves and bed materials. An understanding of these interactions is essential in solving a river-mouth closure problem.

Protection Against Beach Erosion

As mentioned before, the coastal areas are used extensively, and at the same time they are always exposed to wave action. Therefore, beach erosion is becoming one of the most important problems in Japan. The coastal area bordered by the Sea of Japan, especially in the vicinity of Niigata, Toyama, Miyazu and Tottori, are eroded severely by the ocean waves and longshore currents. This action is most severe during the period of seasonal winds in winter. On the coastal areas bordered by the Pacific Ocean, damage to port facilities and ships is caused by the typhoons generated in the Pacific Ocean to the south. It is also believed that the main causes of beach erosion along the southeast coast are these same typhoons. Fortunately, beach erosion damage is not as severe here as it is on the northwest side of the island. Nevertheless, several examples of strong beach erosion can be seen in the areas of Kanagawa, Aichi-Mie, Sennan, Akashi, Kochi, etc. (see Fig. 1).

Construction of Harbors Along Sandy Coastal Areas

Construction of harbors along sandy coastal areas has proven to be a difficult task; this can be exemplified by the many harbors and coastal

structures that have failed primarily due to the builders' failure to consult with coastal engineers. However, the planning and construction of modern ports has been realized recently, a typical example of which can be found at Tomakomai in Hokkaido.

The successes of the new harbors have resulted mainly because the new knowledge of coastal engineering, especially that developed in the United States, has been introduced in Japan; and the number of engineers who study and engage in this field has increased. A large number of papers and reports concerning research and construction works have been published to improve the technique of coastal protection, harbor construction works, etc.

Since 1950, several Conferences on Coastal Engineering have been held in the United States. The proceedings of these conferences have had an important influence on the Japanese engineers, and a Committee of Coastal Engineering was organized in 1955 as one division of the Japan Society of Civil Engineers. A conference held in 1954 in Kobe has been termed the first Conference on Coastal Engineering in Japan. Since then, conferences have been held annually in November. The proceedings published at these conferences have been widely distributed in Japan, and the resultant instruction of engineers has been quite effective.

As one of the Japanese administrative laws, the Coast Act was legislated and came into effect in 1956. As a result of this act, the staff members of the Committee organized a subcommittee for publishing the "Design Manual for Shore Protection Works" in order to give common standards of techniques to the three ministries,—Ministry of Public Works, Ministry of Transportation, and Ministry of Agriculture and Forest—which are each concerned with coastal problems. In preparing the manuscript of this manual, the U. S. Beach Erosion Board's publication, "Shore Protection, Planning and Design" was quite helpful. Beside this, the results of field tests made both in Japan and elsewhere were analyzed to make this manual as reliable as possible in summarizing the present state of the art of coastal engineering.

Japanese Publications

The above discussion outlines the present state of Coastal Engineering in Japan. Following is a selected list of the papers published in Japan to date concerning coastal engineering. Most of these papers are written in Japanese, although some have been translated into English.

Classification of Literature

Coastal engineering is a branch of Civil Engineering that leans heavily on hydraulics, geology, soil mechanics and oceanography. It is probably difficult to divide the field into specific groups, as each of them is closely related to the other, but in this paper the following classification is used for the convenience of the reader.

General

The papers that are written from a general viewpoint concerning the coastal engineering field in Japan are included in this section.

Report of Field Observations

There are many places in Japan where natural adverse phenomena occur; for example, beach erosion, blocking of river mouth, sedimentation in the navigational waterways, etc. Solutions to these problems require much planning and time. For that reason, many engineers have undertaken lengthy field observations and reported the resulting data. Based on these data, many improvements have been planned and actual works have been conducted.

Wind, Waves, and Tides

The main motive element of coastal phenomena is the energy of waves which are generated by the transmission of wind energy to the water through the boundary between air and water. Therefore, the relation between winds and waves is of great interest to engineers. The fundamental characteristics of waves have been studied, in which the dynamic pressure caused by waves occupies an important part. Field observations of waves contribute much toward an understanding of actual wave phenomena; hence, several devices for wave measurement have been developed and actually applied in the field. Finally, more information concerning the Tsunami and abnormal high tides, especially the latter caused by typhoons is needed to help coastal engineers plan adequate structures that may prevent future damages to coastal areas.

Coastal Currents

There are many types of currents in the sea such as ocean current, tidal current, longshore current, etc. One of the most important currents to the coastal engineers is the longshore current. This current is located in the surf zone, moving generally parallel to the shoreline, and generated by waves breaking at an angle with the shoreline.[1] The tidal current through a strait also has an important influence on works designed for facilitation of navigation and protection against beach erosion.

Estuary Hydraulics

In this section, all of the hydraulic phenomena in estuaries are included. There are two representative phenomena, one of which is the density current prevailing in the ordinary flow condition, and the other is the tidal flushing in the areas of high tide range.

Coastal Sedimentation

It is a recognized fact that the amount of sand transported by waves and longshore currents plays an important role in determining the stability or instability of beaches. Hence, the mechanism of coastal sedimentation should be solved in order to improve the techniques of coastal engineering. In this section the reader can find the reports concerning field observations and experimental research on the subject of sand drift.

Coastal Structures

There are many types of structures constructed along the coastal areas. For example, the seawall, breakwater, jetty, groin, etc. are included.

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Miscellaneous

The papers that cannot be classified under any of the above headings, but which are more or less related to coastal engineering, are listed in this section.

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Proceedings of the American Society of Civil Engineers

RINCON OFFSHORE ISLAND AND OPEN CAUSEWAY

John A. Blume,[1] F. ASCE and James M. Keith,[2] M. ASCE

SYNOPSIS

This paper presents the design problems and the construction techniques involved in creating a man-made island of sand, rock and precast concrete armor in the Pacific Ocean offshore from California. This oil production island with the open causeway which connects it to the shoreline constitutes one of the most unique marine installations in the world. The design included many alternate economic studies, model tests in a wave laboratory, and storm damage and wave runup studies with alternate armor types, materials, densities and slopes. The field operations included skin diving and the use of special fathometers in control operations for underwater placement.

INTRODUCTION

The State of California, through its Lands Commission, called for competitive bids in 1954 for the exploration and development of and the production of oil and gas from an offshore area of 1,175 acres called Rincon Lease. This submarine land lies offshore from existing production walls located on piers constructed many years ago. The oil company bidders were to provide all necessary installations at no cost to the State. Offshore facilities had to be in accordance with the then existing requirements and court rulings which essentially specified "solid man-made islands of natural materials".

Richfield Oil Corporation was pronounced the successful bidder since it offered greater oil royalties to the State than any of the many other oil companies that bid. After some legal delays, Richfield was awarded the lease and told to proceed. The engineering firm, John A. Blume & Associates, Engineers, which had already performed various preliminary offshore studies as consultants to Richfield, was in turn told to proceed with the engineering

Note: Discussion open until February 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2170 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. WW 3, September, 1959.

1. Pres., John A. Blume & Associates, Eng. San Francisco, Calif.
2. Project Eng. John A. Blume & Associates, Eng. San Francisco, Calif.

phases of the project except those pertaining to oil exploration and production which were to be done by the client. This was the start of a project which not only developed considerable "romantic" appeal to the public and in the press before completion in 1958, but also included new techniques, storm risks without precedent, and unusual economic considerations in marine and offshore construction.

Project Location and General Description

Rincon Lease is located offshore between Santa Barbara and Ventura as shown on Figs. 1 and 2. Rincon Island No. 1 was located by the client within

FIG. 1

the lease area to provide for maximum production from the greatest area at the least total cost of installation, drilling, and operation. This was in itself an appreciable engineering problem but one that cannot be included herein. With modern slant drilling techniques, the first or "mother" island as located and with conductor pipes for sixty-eight possible wells will develop an appreciable amount of the entire lease.

The water depth at the island ranges from 41 to 48 feet referred to MLLW as datum, from the most shallow to the deepest toe of the island. Tidal range

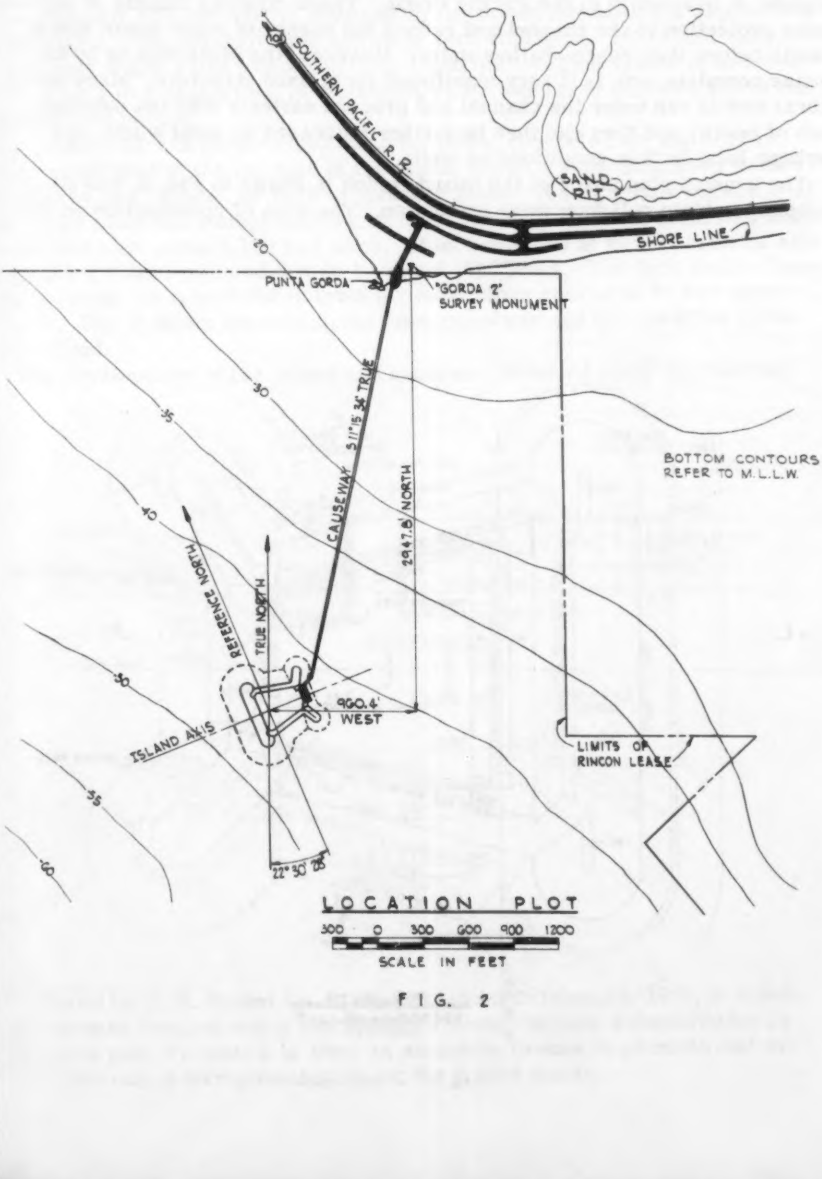

FIG. 2

in this area is as follows:

Extreme High	7.50
MHHW	5.40
MSL	2.58
MLLW	0.00
Extreme Low	-2.5

Although Rincon Island is located in the Santa Barbara Channel which lies between the mainland and the natural offshore islands known as the Channel Islands, it is actually in the Pacific Ocean. These offshore islands do provide some protection to the channel and reduce the energy of many ocean storm swells before they reach shallow water. However, the protection is by no means complete, nor is it very significant for a fixed structure. Many ocean storm swells can enter the channel and proceed easterly with but nominal loss of energy and they can then be further increased by local winds, and perhaps local bottom conditions as well.

The unusual plan shape of the island, which is shown in Fig. 3, was developed to obtain optimum wave protection. The area of construction on the

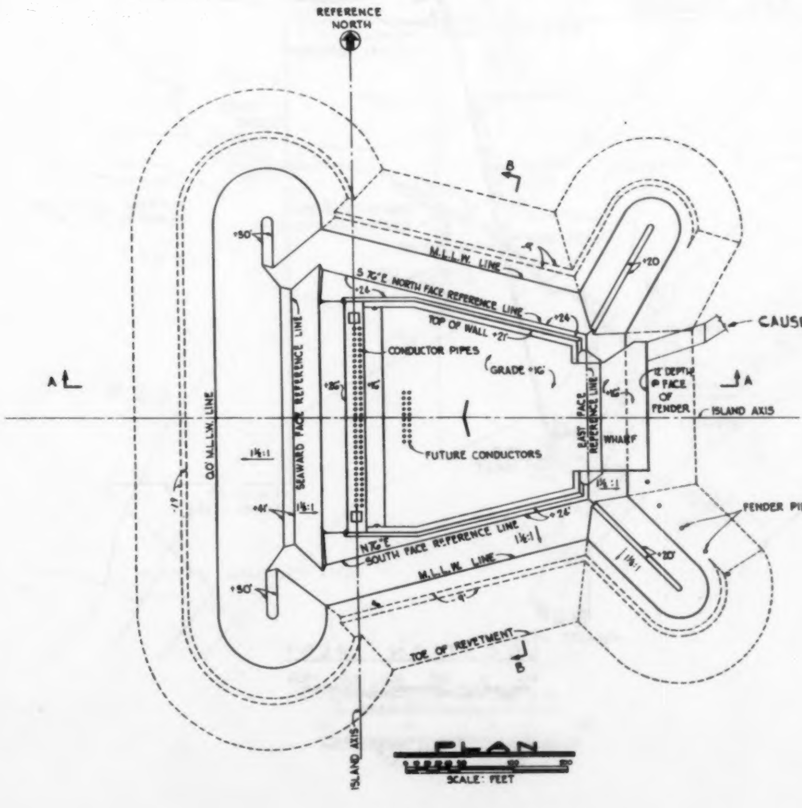

FIG. 3 RINCON ISLAND

ocean floor is about 6.3 acres, at MLLW the area is 3.2 acres, and the gross area at elevation + 16 feet is 2.1 acres. The net usable flat area at this level, exclusive of the wharf, is 1.1 acres, although considerable additional usable space is obtained by effective use of vertical wall surfaces inside the rock armor.

Rincon Island is constructed of rock revetments which contain sand fill. It was constructed in stages (Fig. 4) and contains many types and gradations of rock. The most exposed face is protected with 1,130 concrete tetrapods,* each of 31 tons. The top elevation of the seaward breakwater wall is at + 41, the sides at + 24, and the wharf and working area are at + 16 feet. There are approximately 618,000 tons of material in the island including the 35,000 tons of tetrapods. The exterior side slopes of rock rubble are 1-1/2 to 1 except on the east wings which are at 1-1/4 to 1 and the tetrapod armor is at 1-1/2 to 1. Fig. 5 is a photograph of the island which was taken near the end of the construction period.

A small wharf of prestressed concrete piles, concrete cap, and timber deck is provided at the lee side of the island within a semi-protected harbor created by two "wings" or rock breakwater stubs. A single lane causeway of steel pipe piles and timber decking on steel stringers extends from this wharf to the abutment some 2,730 feet away. Most bents are at 40 feet centers with alternating single-pile and double-battered-pile bents. The deck level climbs sharply from the island and is level for most of its course at 35 feet above MLLW. Fig. 6 shows the island, the open causeway and the coastline in the background.

The development of the island and causeway provided many engineering

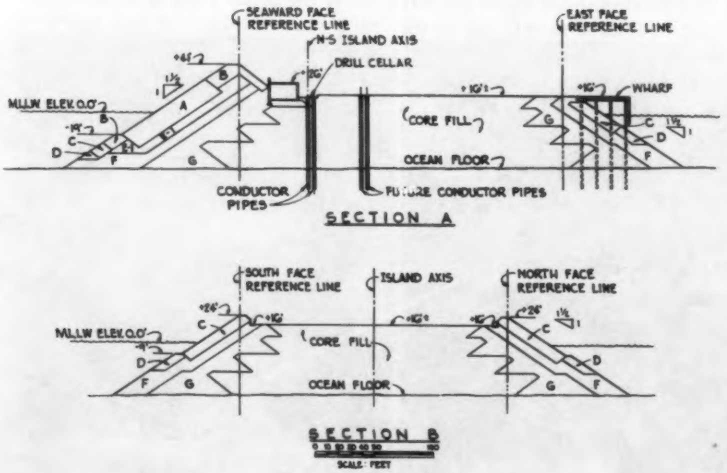

FIG. 4 RINCON ISLAND

*Covered by U. S. Patent No. 2,766,592 issued October 16, 1956, to Establissements Neyrpic which has given Sotramer (Societe d'Exploitation de Brevets pour Travaux a la Mer) an exclusive license to promote and exploit the use of tetrapods throughout the United States.

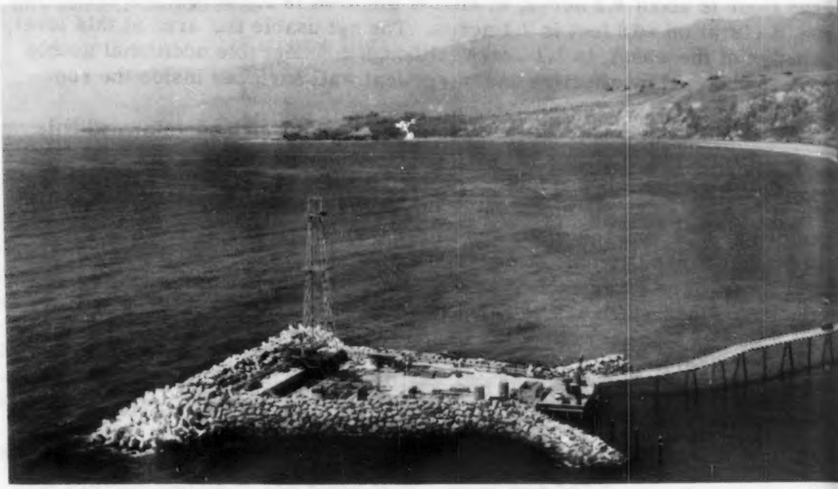

Fig. 5. Aerial View Looking Northwest

problems which required first-time techniques for their solution. It was by no means a simple matter of dumping rock in the ocean. In order to keep costs low and still provide a satisfactory installation with certain anticipated risks, a great deal of engineering study and many "judgment" type decisions

Fig. 6. Aerial View of Island and Causeway

were necessary. The design is not, and could not be, conservative, but it was thoroughly considered.

The Basic Problem

The Owner's offshore lease from the State stipulated that the area should be drilled either from shore, from existing offshore structures, or from a solid island of natural materials. A comparison of slant drilling costs from shore with rough estimates of island costs indicated an island of natural materials was the economical solution. The Owner's geological studies dictated the general location and the basic problem became the design of an economical, permanent island of natural materials suitable for oil well drilling and production. Moreover, the installation was not to detract from the natural appearance of the coastal area.

The size of the island was to be determined by operational area requirements plus allowance for armor layers and their necessary side slopes. These factors in turn were functions of the optimum number of oil wells on the island, the production functions to be done on the island versus on shore, ocean swell heights, periods, and many other considerations. Many investigations and alternate economical considerations were conducted by the Owner's production department and by their engineering consultants working in close collaboration.

Site Investigation

Bottom contours were obtained from existing maps supplemented by lead line soundings and fathometer runs with one of the Owner's exploratory drilling ships, the "La Ciencia". Soil borings of the ocean bottom were made from the La Ciencia by a variety of methods with two primary purposes. One purpose was to determine the suitability of the ocean floor as a foundation for the island, and the other to determine if a satisfactory source of dredger fill material for the island core was available within economical pumping distance. The simplest sampling device was a "snapper" or small spring loaded clamshell for obtaining samples of the surface material on the ocean floor. The "dart" sampler was a heavily weighted stabbing device which, when dropped to the ocean floor through the water, would recover a cylindrical sample up to three feet in length. A jet-churn rig was used to recover cylindrical samples from various depths by jet-churning to the desired depth and then stabbing samples from the bottom of the hole. This last rig was later replaced by a rotary rig which colud also obtain cylindrical samples by drilling to the desired depth and then stabbing the sample from the bottom of the hole. The first two methods of sampling were used in the search for dredger fill material and the latter two for deeper information near the island site.

Bottom conditions vary uniformly throughout the lease area. Overburden material on the ocean floor is a silty sand ranging into sandy silt, and increasing in thickness as the water depth increases. At the island site it ranges from 14 to 25 feet in thickness. The average slope of the bottom at the island site is 3%.

Table I indicates some of the materials encountered. Underlying the overburden is a geologically recent shale or "siltstone" formation. Consolidation tests on samples of the overburden material indicated a probable

Table I

Typical Overburden Materials at Island Site

Sample depth	surface	5'	12'	18'	24'
Dry wt., lbs./cu.ft.	80	98	99	97	94
Wet wt., lbs./cu.ft.	101	123	123	119	117
Percent moisture	27	26	25	23	25
Percent passing following U. S. Std. sieves					
No. 30	99.7	100.0	100.0	99.8	99.8
No. 50	99.5	-	99.8	-	-
No. 80	99.2	99.8	99.6	99.7	99.5
No. 100	98.9	99.6	99.3	99.5	99.1
No. 140	96.8	98.8	97.6	98.0	95.1
No. 200	59.6	76.6	81.2	80.6	61.3
No. 270	30.9	37.1	56.4	52.9	37.9
Hydrometer tests effective particle size					
.050 mm.				46.0	
.037 mm.				32.5	
.024 mm.				23.0	
.017 mm.				19.1	
.010 mm.				15.9	
.0072 mm.				12.7	
.0050 mm.				11.1	
.0035 mm.				9.5	
.0027 mm.				6.3	

settlement of less than six inches at the ocean floor from the weight of a solid island, most of which settlement should occur during construction. Bottom material shoreward of the island was not as coarse as desirable for dredger fill, but studies indicated that proper control of the discharge location could utilize the ocean currents to separate and waste the fine fractions to leave a satisfactory granular core of dredged material.

Faults are known to exist near the island site, some of which are active and most of which are not. The Santa Barbara area has had appreciable earthquakes in recent decades.

The coastline in this area has appreciable littoral drift and sand transportation. However, the sand movement at the location and in the water depth of the island is negligible. Moreover, the island is so small and so far from the coastline as to have no effect on coastal sand erosion or accretion. The possibility of the loss of existing materials adjacent to the island was, of course, considered. Indications were that some minor changes in natural bottom deposits had occurred in recent years and would again occur.

Oceanographic Studies

Wave forecasts for the island site were prepared in considerable detail, and covered estimated heights, periods, direction and frequency of occurrenc

Basic data for the wave forecasts were compiled from several sources. Available wave measurements from wave recorders and from trained observers, hindcasting from synoptic weather maps, and past records of severe storms were all utilized. Refraction studies were then required to tailor this information to fit the partially sheltered position of the island. The Channel Islands and the westward trend of the coastline as far as Point Concepcion serve to protect the island site from many, but by no means all, of the Pacific's winter storm waves. The Santa Barbara Channel offers an inviting approach of almost unlimited fetch from the west right up to Rincon Island for the less frequent but still probable storm waves which approach from this critical direction. This "partial" protection serves to confine the approach of really large waves and had considerable influence on selecting the odd configuration of the island as well as its orientation.

The wave studies were also concerned with lower wave heights from all directions, which, though less dramatic still had an influence on the island design. Frequency of occurrence of these lower wave heights is especially important in planning and scheduling marine operations in exposed locations. Figs. 7 and 8 illustrate the manner in which this information was presented in the island bid documents.

Rock Sources

The closest developed quarry site convenient to marine loading facilities was on Catalina Island. Since this represents a barge haul of approximately

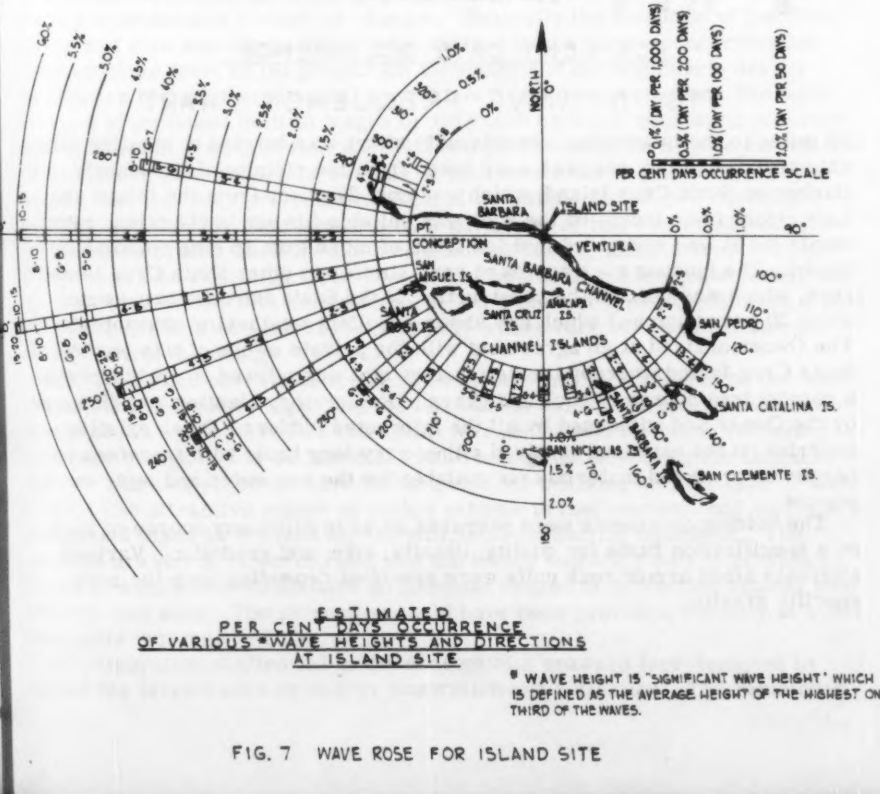

FIG. 7 WAVE ROSE FOR ISLAND SITE

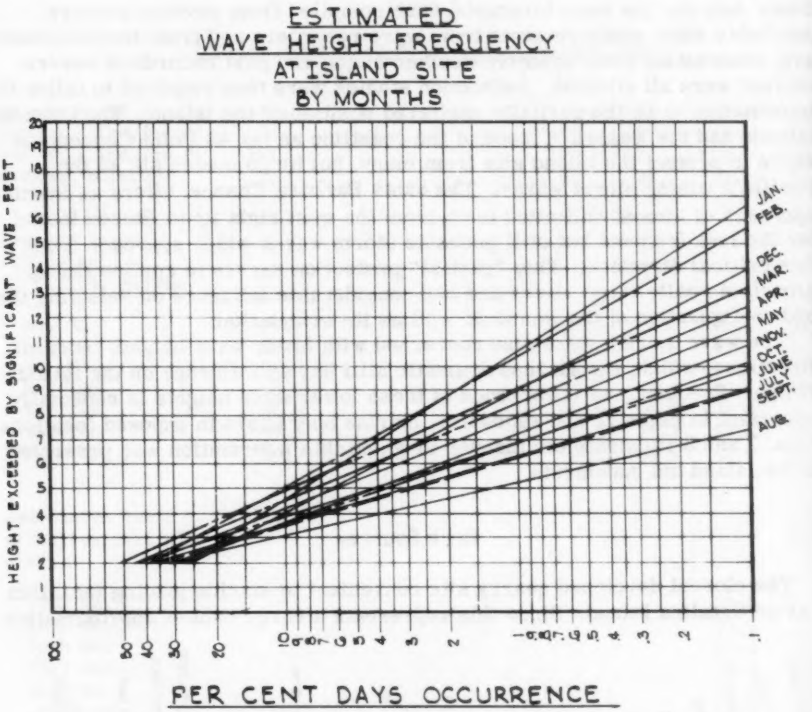

FIG. 8 WAVE FREQUENCY CHART

90 miles to the island site, considerable effort was devoted to locating alternate sources. Rock samples were taken from the vicinity of Prisoner's Harbor on Santa Cruz Island, which was only 27 miles from the island site. Laboratory tests indicated the rock was suitable for use in the island revetments but it was estimated that few units of more than 15 tons could be obtained. The igneous rock appeared very similar to other Santa Cruz Island rock, which was used in the construction of the Santa Barbara breakwater about 30 years ago and which has shown excellent weathering characteristics. The Owner arrived at an agreement with the private owner of this portion of Santa Cruz Island whereby the new quarry site was offered to all bidders as a royalty free source of rock and gravel. Exploratory blasting was financed by the Owner and witnessed by all the interested bidders. Other existing quarries on the mainland involved either very long hauls with transfers to barges or produced materials not suitable for the sea water and wave exposure.

The bidding documents were prepared so as to allow any source of rock on a specification basis for quality, density, size, and gradation, Various alternate sized armor rock units were specified depending upon the rock specific gravity.

Precast Concrete Armor

Studies of the use of precast concrete armor revealed several factors which made it desirable to provide for the use of precast concrete armor as an alternate to the use of large rocks for the class A or largest rock category. Recent research in the use of tetrapods showed that for a given weight and specific gravity they have more stability against wave action than quarried rocks. Practically, this meant that lighter units would be required for the seaward face of the island and hence a smaller crane would be required for placing the armor. Equally important was the lessened risk to any contractor planning to open a new quarry site for the island project by elimination of the necessity for producing and transporting very heavy rocks. It is usually difficult to predict what maximum size of rock it is possible to produce economically from a quarry site until actual operations are under way. In addition to tetrapods, the design included tetrahedrons as optional precast concrete armor.

The alternate of using precast concrete armor also made it practical for a contractor to build the island from an onshore quarry since the next maximum required size of quarried rock, the B grade, could be hauled over public roads with normal equipment. The various sizes of the tetrapod Class A armor and other armor rock classifications are shown in Table II.

Preliminary Island Designs

During the preliminary design stages many elements of the island underwent a considerable number of changes. Basically the evolution of the final shape and size was the result of joint studies by the Owner concerning his requirements from an oil production viewpoint and the engineer's design search for the most economical way to meet such requirements. The basic scheme of an island built in stages or lifts with each lift consisting of a rock dike containing a core of fine material and protected externally be revetments of heavier armor materials was one of the few elements that remained constant. The partially protected location of the island introduced seemingly endless possible configurations to take advantage of this circumstance. The oil industry prides itself on close economical design, and the Rincon Island Project was no exception. The Owner wanted a safe and adequate island, but not one gold plated with safety factors. On a pioneer type project such a goal is rather difficult to achieve, particularly where rare but possible storms could control the design.

Early studies showed an open causeway or roadway trestle to be the most economical method of supplying the island both during and after construction. However, in order to reduce the initial investment and for other reasons preliminary design had to be based on constructing and supplying the island by water. Various schemes were investigated for utilizing an LCT type of landcraft. The attractive aspect of such a scheme is that conventional oil field equipment could be utilized for the entire oil production operation. The serious drawback, however, was that the island was too small to provide a landing ramp which would have an adequate degree of protection against moderately bad seas. The protection could have been provided, but only at a considerable increase in cost.

Configuration studies led to the concept of a seaward face designed to resist the large waves by energy absorption and reflection. The remainder

Table II

ARMOR WEIGHT REQUIREMENTS
for
VARIOUS SPECIFIC GRAVITIES

CLASS A TETRAPODS

Sp. Gr.	Solid Wt. per cu. ft. lbs.	Min. Wt. of Each Unit Tons	Thickness of 2 layers ft.
2.3	143.5	38	18.3
2.4	149.8	31	16.9
2.5	156.0	26-1/2	15.0
2.6	162.2	22-1/2	14.7
2.7	168.5	19-1/2	13.9

ARMOR ROCK

Sp. Gr.	Solid Wt. per cu. ft. lbs.	Class	Min. Wt. of Each Unit Tons	Min. Avg. Wt. of Unit Tons	Thickness of 2 layers ft.
		B	29	32	15.4
2.25	140.4	C	12	13	11.5
		D	6	6-1/2	9.1
		B	23	26	14.2
2.35	146.6	C	10	11	10.6
		D	5	5-1/2	8.4
		B	20	22	13.2
2.45	152.9	C	8-1/4	9-1/4	9.9
		D	4-1/4	4-1/2	7.3
		B	17	19	12.4
2.55	159.1	C	7	7-3/4	9.2
		D	3-1/2	4	7.3
		B	14	16	11.6
2.65	165.4	C	6	6-3/4	8.7
		D	3	3-1/2	6.8
		B	13	14	10.9
2.75	171.6	C	5-1/4	5-3/4	8.2
		D	2-3/4	3	6.4
		B	11	12	10.3
2.85	177.8	C	4-1/2	5	7.7
		D	2-1/4	2-1/2	6.1
		B	10	11	9.8
2.95	184.1	C	4	4-1/2	7.3
		D	2	2-1/4	5.8
		B	8-1/2	9-1/2	9.2
3.05	190.3	C	3-1/2	4	6.9
		D	1-3/4	2	5.5

of the island would be shadowed by the seaward face and the other revetments could be much lighter than the seaward face. This concept led to an investigation of the possibility of first building the seaward face as a complete breakwater and then constructing the rest of the island in the lee afforded by this first stage of construction. This initial stage was such a large proportion of the total island, however, that the savings in materials by building the seaward face integral with the island definitely outweighed the greater ease of construction. Moreover, the initial breakwater would not have provided complete protection. Another interesting scheme given considerable study and even model tests was the feasibility of sinking ship hulls as the basic core of the seaward face or, alternatively, using them as submerged breakwaters seaward of the island proper. Surplus ship hulls were then available on the west coast at attractive prices.

Island Model Studies

As is typical of many hydraulic design problems, several elements of the design could best be checked by model tests in a laboratory. The model tests for the island involved two series of wave tests. The first of these was a three-dimensional model test in a basin to check the configuration of the island and the second series involved two dimensional models in a wave channel. In order to keep costs within a limited budget and still obtain a maximum amount of information from the test, movie films were made of most tests and time consuming measurements were kept to a minimum.

At one stage of the design the owner wanted a small concrete slip about 40 feet by 150 feet on the leeward side of the island for use as a small boat harbor for servicing the island. One purpose of the three dimensional model was to determine what degree of baffling would be required to maintain quiet water in the slip during stormy weather. As was expected the model showed the slip was highly resonant to wave periods typical of Pacific storms and only a water tight door or lock would provide quiet water by baffling. An alternate solution of providing short stub breakwaters at each of the eastern corners of the island was so effective in maintaining quiet water during most wave conditions that the slip was replaced by a small wharf on the eastern side of the island. In general the other configuration features of the island were found to be satisfactory as was the concept of a high seaward face sheltering the tapering work area of the island which is at a lower elevation.

Using the same model, the feasibility of using two concrete ship hulls as a separate, submerged breakwater seaward of the island was investigated. By comparison of the wave runup on the sides of the island, the effectiveness of different locations and spacing for the hulls was studied. Although cost studies indicated possible appreciable savings in construction cost with use of the hulls, the owner elected not to use them because of the less attractive appearance and possible adverse public relations for the oil industry. Fig. 9 indicates a three dimensional model test.

The second series of laboratory tests involved two dimensional tests of the proposed seaward face revetment section in a wave channel. Fig. 10 illustrates a model section being subjected to a wave seven feet higher than the design wave. The first test runs were of a wave height slightly below the design wave height of 27 feet. These runs verified the stability of the design for the design wave. Following these initial tests, the wave heights were increased in steps to a maximum height of 34 feet.

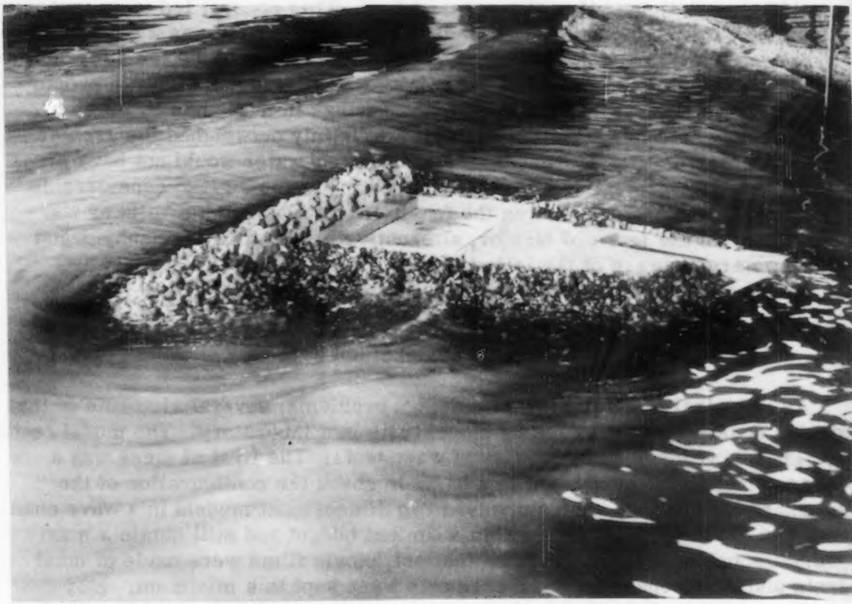

Fig. 9. Three Dimensional Wave Action Test.

As anticipated, the section showed considerable distress under attack of waves appreciably larger than the design wave. The gratifying feature, however, was that the section showed no tendency towards a catastrophic type failure due to any single wave, but rather a gradually increasing distress. This was consistent with the basic design objective of an economical section which might sustain damage but which would not endanger the whole island when subjected to the rare occurrences of the very large waves.

Island Economic Studies

The first preliminary studies made it clear that it was not economically feasible to design the seaward face revetment so that it would be completely stable against all possible storms. This fact is well illustrated by the history of conventional breakwaters along the Pacific Coast. All deep water breakwaters with a severe exposure are expected to and generally have suffered occasional damage. The dire consequences of a complete failure for the island's revetments made the problem much more critical than for breakwater design. A rational approach to the island problem was developed which essentially consists of six steps: (1) prediction of frequency of occurrences of large storm waves at the site; (2) correlation of predicted storms with laboratory tests of revetment sections; (3) estimates of cost and damage for various trial designs; (4) evaluation of damage to the various designs; (5) economic analysis of various designs; and (6) selection of final revetment sections.

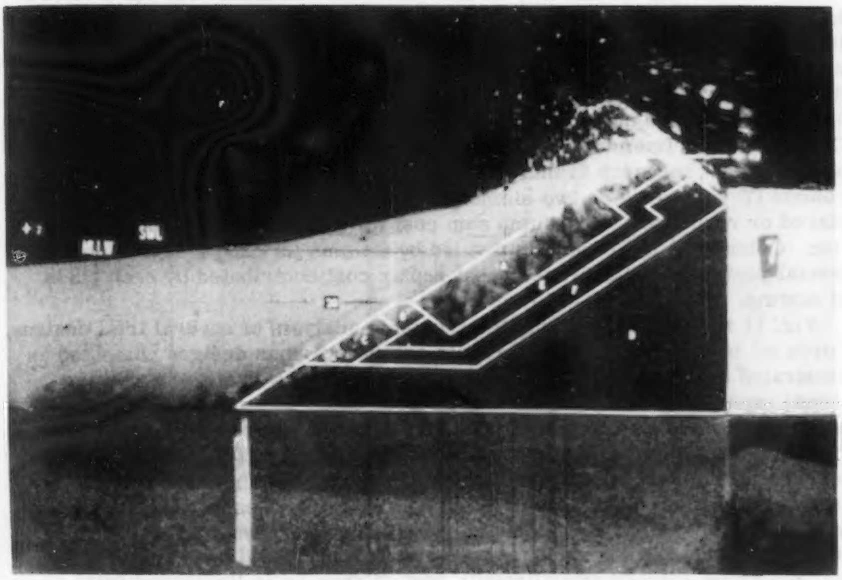

Fig. 10. Seaward Face Test with 34-Foot Wave

Table III summarizes the calculations for estimating the average annual repair cost for a trial design of the island's seaward face revetment. Columns (1) and (2) were developed by the oceanographic study of the island site. For column (3) the maximum wave of a storm is assumed to be 1.9 times the significnat wave of the storm. Column (4) correlates the storm waves and laboratory waves.

It was assumed that the maximum wave of a wave train best describes the

Table III

Estimated Annual Repair Cost for 27 ft. Maximum Wave Design

(1) Significant wave height of storm ft.	(2) Frequency Storm recurs once in years shown	(3) Max. Wave height in storm ft.	(4) Equivalent wave ht. for use with armor formula ft.	(5) Ratio max. wave ht. to max. no damage wave	(6) Estimated % damage to armor per storm	(7) Estimated cost of repair for storm $	(8) Incremental avg. annual repair cost for storm $
14	14.4	26.6	23.0	.96	nil	-	-
15	23.8	28.5	24.3	1.01	2	81,500	3,420
16	43.5	30.4	26.2	1.09	7	130,000	3,000
17	74	32.3	27.9	1.16	13	208,000	2,810
18	141	34.2	29.5	1.23	23	286,000	2,030
19 or over	107	36.1+	31.2	1.30	35	400,000	3,740

Average annual repair cost = $15,000.

destructive ability of the wave train and that the maximum wave of a laboratory wave test is 1.16 times the laboratory designated wave height for the test. Based on these assumptions a wave height 1.64 times the significant wave of a predicted storm was used in the required armor weight formula to determine the armor for a no damage design for that storm. Columns (5) and (6) were based on damage estimates obtained in the wave laboratory tests. Considerable refinement of such damage estimates is now possible as a result of recent research in this field.(1) The estimated cost of repair in column (7) was based on two elements: a unit price for armor materials replaced or recovered, and a lump sum cost for mobilization and demobilization. Column (8) is Column (7) divided by Column (2) and gives the incremental portion of the average annual repair cost contributed by each class of storms.

Fig. 11 is a plot illustrating the economic analysis of several trial designs. Curve (a) is the average annual repair cost for various designs computed as illustrated by Table III, and curve (b) is the present worth of the average annual repair cost for a 25 year period at an interest rate of 6%. Curve (c) is the estimated construction cost of each of the various trial designs. The capitalized cost, curve (d), is the sum of curves (b) and (c), and its low point represents the most economical design. Fig. 12 presents another method of analyzing the same information. Starting with a trial design which is definitely less stable than the most economic, curve (a) represents the additional investment required for various more stable designs and curve (c) is the corresponding average annual repair cost. Curve (b) is the incremental reduction in average annual repair cost. Curve (e) is curve (d) divided by curve (b) and represents the effective return on each incremental investment. The economical design is selected as the one beyond which an additional increment of investment fails to offer an attractive return.

It is realized, of course, that such procedures involve low, if not negative safety factors under extremely adverse conditions. However, it is a logical philosophy for severe but infrequent conditions like destructive earthquakes, bomb blast, or severe storm waves to design to high unit stresses and deflections in order to absorb energy and save the construction cost for materials which may never be needed. However, catastrophic-type failures must be considered and avoided in such calculated risk designs.

The Final Island Design

Figs. 3 and 4 show a plan and general sections illustrating the final design of the island. This design evolved from five basic inter-related problems; the island size and shape, the revetments, the filter and core, the general scheme of construction and, of course, the cost. Size requirements were established by the Owner to provide the necessary space for oil drilling and production facilities consistent with the anticipated drilling and production program. The final shape was developed from the oceanographic, model, design, and economic studies. The west, or seaward, face is designed to withstand the heavy seas from the winter Pacific storms, and partially shields the remainder of the island. The north and south faces, or sides, of the island are designed for 12 foot waves, since the maximum size from these directions is limited by the fetch inside Santa Barbara Channel. The east face, or shore side of the island is provided with a small wharf protected against ocean storms by the northeast and southeast stub breakwater or

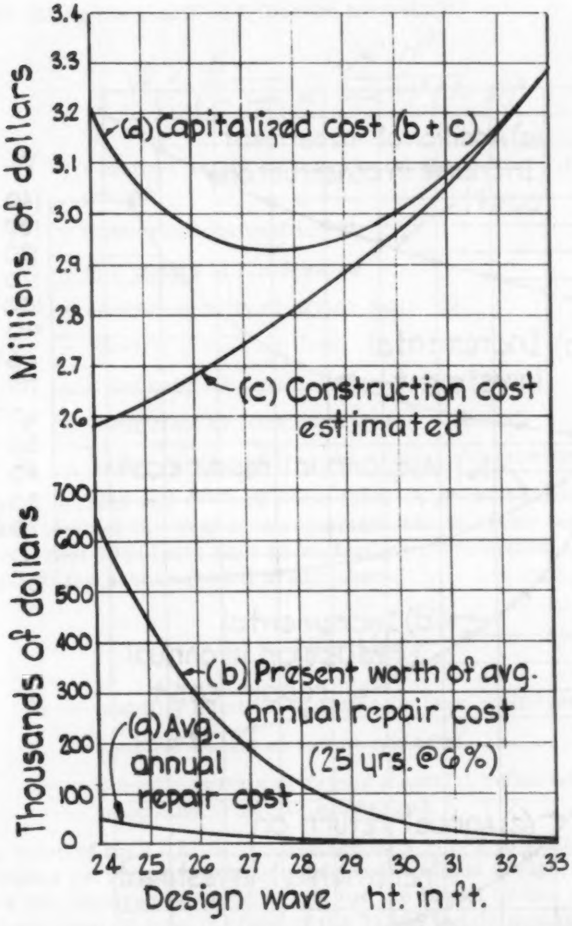

FIG. 11 COMPARATIVE ECONOMY OF DESIGNS

"wings" as they came to be called. The island was originally designed to be served by the wharf only (no causeway) for reasons previously outlined.

The revetment design for the west face also included a cellular wall structure adjacent to the double line of conductor pipes. This structure serves as a support platform for the drill rig which straddles the drill cellar. The rig can be skidded in a north-south direction to center over the desired well. In addition the cellular wall structure serves as a backstop or secondary line of defense for the west face revetment. This revetment can thus sustain considerable damage before the wells themselves could be exposed to direct wave attack. This west face revetment is designed to be stable against 27 foot waves.

The required weight of tetrapods was determined by extrapolation of the U. S. Waterways Experiment Station tests for the Crescent City breakwater

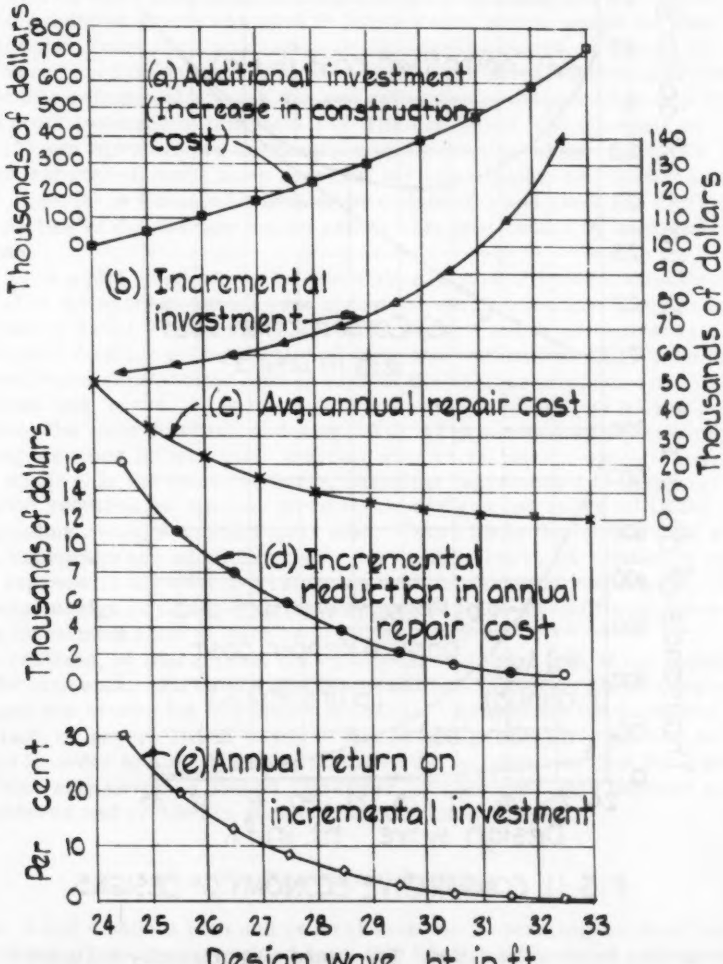

FIG. 12 ECONOMIC ANALYSIS OF INCREMENTAL INVESTMENTS

design.[3] Using the modified Iribarren formula.[4]

$$W = \frac{K' w\, S_f\, S_r\, \mu^3 H^3}{(\mu \cos \alpha - \sin \alpha)^3 (S_r - S_f)^3}$$

where

W = required weight of individual units of armor material in pounds

K' = dimensionless coefficient

w = unit weight of fresh water

S_r = specific gravity of armor material

S_f = specific gravity of fluid

H = wave height for no damage

μ = coefficient of friction of armor material

α = angle, measured from horizontal of breakwater slope

values of $K' = .0223$ and $\mu = 1.10$ were used. A prime reason for the wave laboratory tests of the west face revetment was to verify these values.

More recent 1958 tests have given further confirmation. Using the WES formula[1,5] for rubble-mound breakwaters.

$$N_s = \frac{\gamma_r^{1/3} H}{W^{1/3} (S_r - 1)}$$

N_s = stability number or dimensionless coefficient

γ_r = specific weight of armor material

Sr = specific gravity of armor material referred to the water in which the armor is submerged.

The value of $N_s = 2.37$ for no damage on a 1-1/2 to 1 slope gives required unit weights very close to those used for the island design. For the same criteria the required tetrapod weight for the island by use of formula (2) would have been 30 tons in place of the 31 ton weight actually used. Since the design is not for the maximum wave predicted, the slight advantage is quite acceptable.

The seaward face height of + 41-feet above MLLW was selected to limit the overtopping from a 34-foot wave to an approximate height of 3-feet. As shown in Fig. 4, five classes of armor are utilized in the west face revetment. The heaviest is Class A for which the Contractor selected the option of 31 ton concrete tetrapods having a specific gravity of 2.40. The bid documnets actually offered numerous options for each of the five classes or revetment armor because no single developed source of rock material was definitely more advantageous than others. The individual minimum weight requirements for Class A, B, C, and D rock were allowed to vary with the specific gravity of the rock. In addition either tetrahedrons or tetrapods of precast concrete were optional in place of rock for Class A armor. Since variations in size also varied the thickness of armor layers, the quantities of all revetment materials and the core varied with the different options. In all, 59

optional quantity tabulations were included in the bid documents. This allowed bidders to evaluate the advantages of sources of high specific gravity rock which would make considerable reduction in the individual weights of armor.

When all factors except specific gravity are constant the required weight of individual units of armor material, W, reduces to the following:

$$W = \frac{C\,S_r'}{(S_r' - 1)^3}$$

where

c = a constant

Table II illustrates the variation in armor rock with specific gravity for equivalent designs as computed on the basis of the above formula.

An effective filter is essential to avoid the loss of core material through the rock layers from "pumping" caused by wave action. The filter could not conform with the generally accepted "T-V"(2) gradings recommended by Terzaghi and modified by the Waterways Experiment Station at Vicksburg. Since it was considered impractical to place the relatively thin blankets envisaged by the T-V gradings in an exposed ocean location, the size spread in each material was made considerably greater than recommended. It is anticipated that there will be minor losses in the filters, especially the Class G material, as the fines are lost from the outer layers and a stable grading is achieved. Most of this readjustment is believed to have occurred during the construction phase as the Class G material was normally the first material placed in each lift. The Class F material actually served a dual function. In the lower layers it was used as the lightest class of armor and elsewhere as the outer layer of filter. Class F material was a quarry run material with an open gradation ranging from four tons down to a minimum of 15 percent less than 5 pounds. Class G material was an optional quarry run or gravel material with a dense gradation ranging down to not less than 25 percent passing the No. 20 sieve. The core was sand for the reason that this was less costly than even a quarry waste. The bidders were allowed the option of placing the core by dredger from borrow areas on the bottom or to import from shore borrow.

Island Construction

Sealed bids on a unit price basis using the Engineer's quantity estimates for total cost comparison were obtained from selected contractors. In August, 1956 the contract was awarded to the low bidder who elected to open his own on-shore quarry about 6 miles from his loading-out site and to use precast concrete tetrapods for the Class A armor. At the loading-out site, located 4-1/2 miles upcoast from the island site, the contractor built a temporary loading structure in approximately 22 ft. of water which consisted of an L shaped pier of eight forty-foot diameter steel caissons filled with rock and sand with a trestle connection to shore. The pier was sized to provide moderate protection for one flat barge. A 50-ton stiff leg derrick was mounted on the pier to handle materials. Fig. 13 is an aerial photo of the contractor's construction yard and loading out pier.

Tetrapods were cast in the construction yards on shore adjacent to the loading out pier. Since locally available sand and aggregates are mildy re-

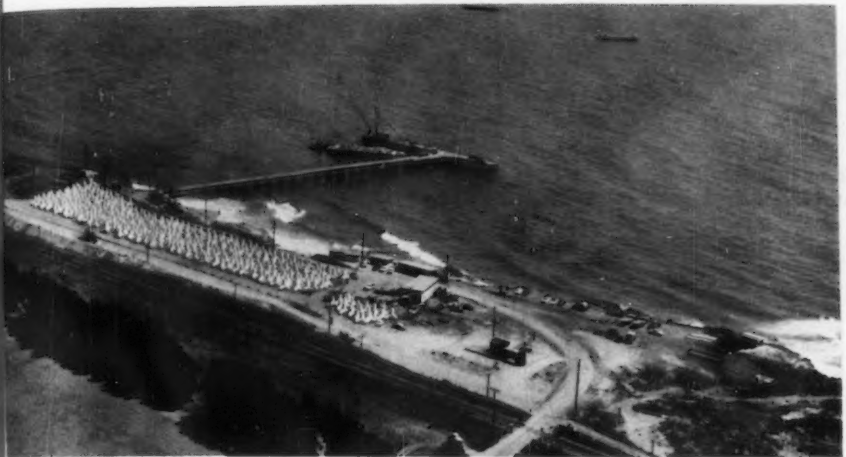

Fig. 13. Contractors Construction Yard and Loading-Out Pier

active, the cost of obtaining non-reactive sand was investigated. A type II low alkali cement had been specified. Recent production from the selected cement mill had been averaging approximately 0.3% alkali calculated as equivalent sodium oxide. It was decided to use the locally available sand and aggregates, but to maintain a close check on free alkali content of the cement. The job average was 0.29% with a range from 0.20% to 0.48%. The concrete mix used 5 sacks of cement and 3 in. maximum size of aggregate. A calcium lignin sulphonate additive was used at the contractor's option. A 27 E paving mixer operated on a bulkhead ramp adjacent to the casting pit so that its bucket could discharge directly into the tetrapod forms which were of two piece steel construction. The bottom section formed the bottom half of the three lower legs, and the top section formed the top half of the lower legs and the upstanding leg. End gates for the bottom legs were hinged to the top section. The contractor used 36 bottom sections and 12 top sections, which allowed for a pouring schedule of 12 tetrapods per day. The top forms were stripped after 20 hours and the tetrapods were removed after three days. For this first lift a special compression sling, developed by the contractor, gripped the tetrapod by pressing a bearing plate against the flat end of each bottom leg. By using this sling (Fig. 14) the concrete was in compression and, although the concrete was still green, the tetrapood could be handled without damage or overstressing. A large crawler crane lifted the tetrapods from the casting pit and placed them in the adjacent storage yard to complete their curing.

The first material for the island was placed in February, 1957 after several months of quarry development and the construction of temporary facilities. The majority of the marine work was done on a two shift, six day work week since the marine equipment charges represented a sizable proportion of the contractor's costs. The general procedure was to build the exterior rings of each lift of Class G and F, then place the core material and armor rock. All F, G and core material below elevation-15 were placed by bulldozing the material over the side of carefully spotted barges. Armor materials were placed by cranes. The contractor placed wood pile dolphins on

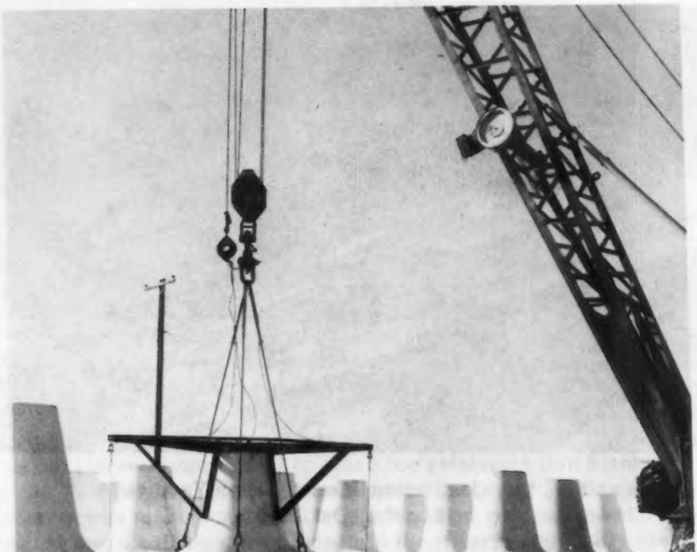

Fig. 14. Handling Tetrapods

the north and east sides of the island work area. Targets strung between the dolphins were used to provide position lines for placing the below-water materials. The island first broke water in October, 1957. The seaward face was then carried to elevation ± 17 ahead of the other sides in order to provide some protection from the approaching winter weather. Before complete closure of the island above water, sufficient core was placed on the south side to allow the barge mounted crawler crane to be unloaded by beaching the barge against the core and walking the crane off on a temporary ramp of core fill. The top lifts of tetrapods and armor rock were placed by the crawler crane on the island. The final closure of the north face was made in January, 1958.

Core fill for the island was a medium to fine sand obtained from the cliff behind Punta Gorda about three quarters of a mile from the island site. The core material was hauled by truck to the contractor's loading out pier, then hauled by barge to the island site. It was not surprising that the contractor elected not to dredge the core fill, since the final design required a relatively small amount of sand for a dredging operation. The lift type of construction required that core sand be placed on an intermittent schedule, and the open

sea is not the ideal place to operate a dredge even on larger projects.

Although two moderately severe storms occurred during January, 1958 when the island was in an incompleted and vulnerable stage, the island received only very light damage. The contractor's loading-out pier was damaged in the second of these storms and required a month for repairs.

The 68 steel conductor pipes were driven when the core elevation was approximately +11. These pipes are the initial casing for the future oil wells to be drilled through the island by the owner and were driven to a penetration of 15 feet into the original ocean floor. Work on the concrete walls on the surface of the island was started after the conductor pipes were all driven. For this work a small batch plant was placed on the island.

Work was substantially completed and the owner took possession in August, 1958.

Quarry Operations and Rock Quality Control

Adequate control of rock quality proved a very difficult assignment, and constant effort by both the Contractor's quarry force and the Engineer's field staff was required in order to insure a supply of rock of adequate quality. The rock specification included two quality tests, the Los Angeles Rattler test and the test for soundness by the use of sodium sulfate which test was considered especially important because of the marine exposure. Rock quality varied widely throughout the quarry site. Much of the rock which was the Cold Water Sandstone Formation, Eocene Age, was of excellent quality, but some deposits were very poor and practically uncemented. There were also many intermediate grades. The quarry site, however, contained a vast amount of good material and was of a type which could be quarried in very large unit sizes where desired.

During the period of initial development the Contractor drove many small tunnels or coyote holes into the canyon sides searching for the most desirable rock. The coyote holes in sound rock were later used for primary blasting. An extensive field testing program was carried out by the Engineer on samples taken from these coyote holes, and an intensive search was made for any quickly identifiable characteristics which would correlate with soundness. Acid reaction, specific gravity, Schmidt hammer reading, color, density, grain size, and microscopic examination were all tried but found unreliable. The final solution was to test each separately identifiable type of rock found in the quarry and to classify each type according to its actual soundness test results. Over 100 samples were required for adequate coverage. In order to speed the test results the Engineer's field office was equipped to perform soundness tests on a continuous basis. Untested portions of each sample were retained and small chips from these were carried in a compartmented box as an aid in quick field identification of the rocks. This method proved effective in the majority of the cases, but a few types of rock, which straddled the acceptance line, remained difficult to classify throughout the job.

Rock quantities were measured for payment by barge displacement. Occasional checks of barge gaging accuracy were made by weighing all loads on truck scales and agreements were normally within 1%. Individual weights of armor stone were normally judged by eye. In cases where doubt existed, weight was checked by truck scales or by measuring the cubage of the rock. As the work progressed considerable skill was developed in estimating rock weights. The importance of rock quantity control and also quality control to

the Owner cannot be over emphasized for a unit price contract in 50 feet of water in the open sea.

Field Engineering

One basic problem for the Engineer who provided engineering supervision and inspection throughout the construction was to insure that the filter zones of the revetment construction were adequately placed and that no chinks in this essential element of the island's defense were left for the remorseless attack of the seas. Another responsibility was to see that the various rock layers were placed within acceptable tolerances. The fact that two-thirds of the island's cost was below water points up the difficulty of these problems. Survey and layout work was basically a contractor responsibility and a high percentage of his marine work was directly or indirectly concerned with performing this task. A lead line was almost constantly in use during all underwater material placement operations.

The magnitude of the survey work made it impractical for the Engineer's staff to check all survey operations. Field inspectors observed and spot checked the contractor's marine survey operations, but considerable reliance was placed on independent surveys of the underwater mounds as placed in the early stages of the work by use of a modern ultrasonic depth recorder. An essential feature of this instrument was the narrow (approximately 6 degree) cone of response, which was necessary in order to adequately delineate the sharp breaks in grade typical of the island form. As anticipated, vertical accuracy of the instrument when properly calibrated was no special problem. The never ceasing problem was maintaining adequate horizontal accuracy in a highly congested work area, on the never-quiet ocean surface. Special accessories for the survey boat "Blu-Isle" were helpful in obtaining the desired horizontal accuracy. The echo sounder was a portable instrument so the transducer was mounted outboard toward the stern. The most versatile method of position control was by taking simultaneous sextant angles on three targets from the boat. To help reduce plotting errors a platform was rigged to overhang the transducer, so that both sextants could be positioned over the transducer when taking position shots. As an aid in obtaining a conveniently large horizontal chart scale, a special high speed chart drive motor was installed in the echo sounder. In addition, "spoiler plates" were rigged to be lowered into the water directly behind each of the "Blu-Isle's" twin propellers. The "spoiler plates" were very effective in reducing the boat speed while still maintaining better than normal rudder control. The slow boat speed was desirable to allow close spacing of position shots and as a further aid in maintaining large horizontal scale on the echo sounder charts. These techniques enabled a limited field staff to take accurate and continuous three-dimensional "sweeps" of underwater construction whenever indicated.

A 1 to 120 scale model of the island (Fig. 15) was built by the Engineer's field staff. Progress on the model was maintained currently with progress on the island, so that it served as an easily understood progress report. While the island construction was below water it was especially useful for visualizing the status of the work, and all those connected with construction of the island watched their efforts reflected in the model with gratifying interest.

SCUBA (self-contained underwater breathing apparatus) diving gear was also utilized by the Engineer's field staff (Fig. 16) for inspecting the underwater portion of the work and this also proved very useful. Although there are

Fig. 15. Construction Progress Model

drawbacks to the use of SCUBA gear, its outstanding advantage is that it allows the engineer to see the object in question with his own eyes. Additional advantages of the SCUBA gear are summarized as follows:

(a) Equipment is relatively inexpensive.

(b) For a diver with limited training and experience it is safer than conventional diving gear, although basic training is still an essential.

(c) Equipment is easily portable so that elaborate preparations for a dive are not necessary.

(d) The diver can get around faster and has greater mobility and flexibility of operations.

Against these advantages the following drawbacks must be balanced:

(a) There is no underwater communication system equivalent in convenience to a helmet diver's phone system. Some of the other drawbacks mentioned are a direct result of this lack.

(b) Unless the water is clear, orientation is more difficult to maintain than using

Fig. 16. Resident Engineer Preparing for Underwater Inspection

conventional deep sea gear. A compass is often very helpful, but is useless if ferrous metal is in the vicinity.

(c) Skin divers should work in pairs for safety.

Voids in the rock materials as placed varied from 40% for the armor rock which was essentially of unifrom size to 30% for the Class "G" material which was of reasonably dense gradation. If losses of the core fill material are ignored, the tonnage placed would indicate about 20% voids in this material, the majority of which was placed under water. A more reasonable assumption of 35% voids in place, indicates that approximately 23% was lost due to ocean currents and wave action.

The Open Causeway

The lack of a commercial harbor close to the island site, the savings inherent in running production and utility lines ashore along a causeway rather than on the ocean floor, the convenience of ready access of truck-mounted oil

field equipment, and numerous other reasons finally caused the Owner to decide on an open causeway to the island in lieu of marine transportation. Traffic density requirements were very light so the design aim was for maximum economy.

Span lengths of 40 feet with alternate single and double pile bents were selected from economic studies for minimum cost. The deck elevation of 35 feet above MLLW is adequate to keep the structure above wave crests. The selected design wave of 25 feet is not the highest possible wave at the site, but it represents a calculated risk based on providing an economic life for the structure. Figs. 5 and 6 show the causeway.

At the time the borings were made for the site investigation of the island, several additional borings were made from the "La Ciencia" along the probable alignment for a causeway. These borings indicated very little overburden above the shale formation from a water depth of approximately 25 feet shoreward. Fathometer runs over the proposed alignment at a later time established the bottom profile and revealed occasional rock outcrops out to a depth of 30 feet. A SCUBA diving inspection of some of these outdrops indicated they were similar to the rock outcrops on shore at Punta Gorda. Although a solid fill causeway was considered for the area shore section, an open causeway all the way to shore was selected so that there would be no affect on the normal littoral drift in this area.

The design vehicle load, which represented the Owner's forecast of the heaviest conventional oil field equipment they would require in their island operation was a tractor-trailer of approximately 34 tons gross. If heavier loads are required at some future time these loads can be handled by barge to the island wharf. Wave forces created the greatest lateral loads, but seismic forces based on 0.08g and wind loads of 30 pounds per square foot were also investigated with certain other load combinations.

All piles were assumed to be fixed below the ocean bottom. The point of assumed fixity varied from five to ten feet, depending upon the type of material at the bottom and the amount of moment induced at the lower end of the pile by horizontal loads. The top supports of the single-pile bents were treated as elastic supports with their reactions taken by the adjacent frame-bents through the superstructure. For expansion, the causeway is divided into three longitudinal sections. Battered pile frames, set longitudinally, provide the necessary support in that direction.

Most causeway piles are subject to breaking waves. A probability study of storm damage resulted in the selection of a 25-foot, 12-second period wave as maximum for design. Each bent was checked for wave forces at high and low tides, since either could control the design, depending, of course, on the distribution of the load vertically.

Many sources were investigated for proper drag coefficients. What appears to be a very logical approach to the problem of waves breaking on piles with relatively small $\frac{D}{H}$ ratios was given by Reid and Bretschneider.(6)

Using the Brekeley-Monterey field data, which consisted of measurements of moments on piles, due to breaking or near breaking waves, the drag coefficient was obtained from the relation.

$$C_D = \frac{M}{\frac{w}{2g} \; DH^2 \, K_{Dm}\left(\frac{S_D}{d}\right)d}$$

where C_D = drag coefficent

M = measured moment at ocean bottom on cantilever pile

w = specific weight of water

D = diameter of pile

H = wave height

K_{Dm} = maximum value of wave force factor for drag effect of pile applicable to nearly breaking waves.

S_D = vertical position of action of total drag force on pile above ocean bottom

d = still water depth

Total forces and their centers of gravity were computed by the same source.(6) The forces were then distributed along the pile in general accordance with Munk. (7,8) Instead of using a smooth curve for this dynamic force distribution, the loading was simplified to an equivalent straight line distribution which gave the same or slightly higher results. For example, a 25 foot 12-second wave breaking in 32 feet of water was found by Reid and Bretschneider's work to have a resultant drag force of 9 kips acting 34.5 feet above the ocean bottom. Munk's smooth curve for velocity distribution gave a resultant of 7.5 kips acting 35 feet above bottom, and the simplified straight line force distribution gave a resultant force of 9.5 kips acting 35 feet above ocean bottom.

The basic uncertainty in the correct value of the drag coefficient and the cumbersome analysis of the pile frames using the more refined smooth curves for dynamic force distribution were deciding factors in selecting the simplified loading diagrams for pile frame analysis.

The net pile diameters were used for determing wave forces. In lieu of using an increased diameter as an allowance for marine growth the Owner intends to include pile cleaning as a routine item in the maintenance program.

Causeway Construction

The bid documents provided for five optional combinations of steel pipe or prestressed concrete piles, with steel or concrete caps, steel stringers and timber deck or 40 foot prestressed concrete slabs. Pile driving provided alternates of driving, driving and jetting, or drilling and grouting in the shale rock. It was presumed that driving might be practical for the steel pipe pile alternate, but that drilling and grouting would be required on the near shore piles if concrete piles were used.

The low bid was submitted for the steel pile, steel caps and stringers, and timber deck option. Before final award of the contract two test piles were driven into rock on shore to determine whether or not it was necessary to drill and grout the steel pipe piles. The two test piles of 1/2" wall 16 inch diameter pipes were successfully driven with a heavy drop hammer. As a result of these tests the option of driving the steel piles was selected, and a lump sum contract was awarded on that basis.

The scheme of construction selected by the contractor was to build a temporary work trestle of his design from which the piles for the causeway were driven by a heavy drop hammer handled by a small crawler crane. Stringer assemblies were shop fabricated and placed by a truck crane operating on the work trestle. The cap connection to the piles consisted of a

stiffened connection plate welded to the bottom flange of the cap which fitted into vertical transverse slots in the piles as shown in Fig. 17. The work trestle afforded ready access for alignment of the piles and cutting the slots and thus made the erection work a simple operation. Construction of the work trestle paced the causeway erection work.

Specified pile penetration for the typical single pile and double pile transverse battered bents was 8 feet into the shale formation or a minimum penetration of 20 feet into other materials and a driving resistance giving not less 45 ton safe load by use of the ENR pile driving formula. Required penetration for the four longitudinally battered bents was 50% greater. Pile sizes ranged from 16 inch diameter, 3/8 inch wall at the shore end to 24 inch diameter, 9/16 inch wall for the deep water longitudinally battered bents. All piling was sandblasted and coated with coal tar enamel. The three expansion joints are semi-insulated as a cathodic protection system will be installed by the Owner. Concrete pipe sleeves were installed at the bottom line on the first nine inshore bents as a precaution against sand abrasion.

Work on the causeway was started in November, 1957 and the first vehicles crossed in July, 1958. The January 26, 1958 storm caused approximately one month's delay by knocking over about 750 feet of the contractor's temporary work trestle. Only the abutment of the causeway had been completed at this time.

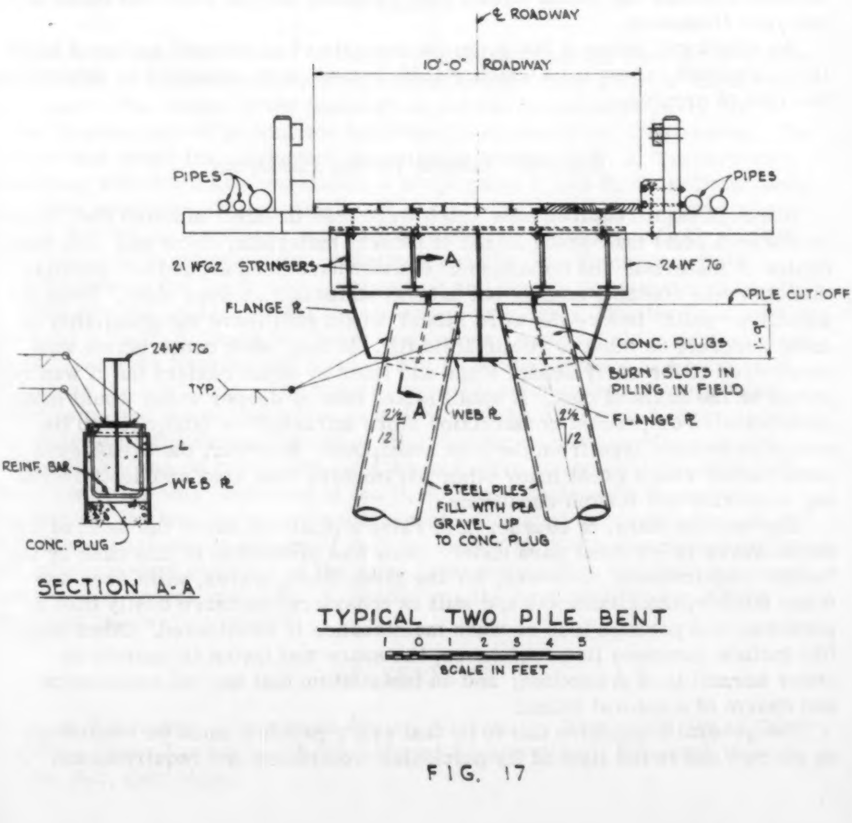

FIG. 17

Controls for Settlement, Erosion, and Damage to Island

A program of controls was prepared for the Owner so that an accurate service record of the island will be maintained. The lack of any precedents of similar structures makes such a program very desirable. Elevation check points are located in several zones, so selected that differential settlements between different zones will isolate the cause of any settlement. If settlements become significant at some future time this knowledge of the cause would be of prime importance in selecting remedial measures. Possible causes of settlement could include in addition to further consolidation of the island and its base due to time and pressure, earthquake or storm wave damage, loss of fine materials because of piping, erosion of rock or tetrapods, subsidence due to drilling or production, or general geological subsidence. None of these are expected to be of major importance in this installation because of the design and construction care and the Owner's pressure oil production techniques.

It is hoped that reasonably accurate wave observations and possibly photographs of large swells can be made by reference to the causeway deck level. Runup or overtopping of the seaward face can also be recorded. This information combined with data on damage, or preferably lack of same, would be an important contribution to marine engineering. To date there has been no damage although the storm swells have probably not yet exceeded those of a ten year frequency.

An additional phase of the program consists of an indexed series of holes in the exposed armor rock which will be periodically measured to determine the rate of erosion.

General: Islands Versus Platforms

Although legal requirements which were then in effect dictated that Rincon Island be a solid man-made island of natural materials, there was still some choice of materials and techniques. It could have been argued for example that concrete contains nothing but natural materials as does steel. Even the adjective "solid" before the word island would still leave the possibility of using concrete or steel to retain solid fill. In fact, such construction was considered in the early design stage and later by some bidders but it was rejected on the basis of cost. A small island and/or deeper water would make prefabricated or precast construction more attractive or compared to the rubble revetment type from the cost standpoint. However, the break-even point varies with a great many other parameters than area and depth including wave size and bottom conditions.

The obvious thing, of course, is to raise a platform above the level of the worst waves to let these pass under. Such was prohibited in this case by the "solid" requirement. However, for the given depth, waves, work area and other factors, the island was and still is considered no more costly than a platform, and perhaps less so when maintenance is considered. Other benefits include complete fire resistance, the space and layout to operate as under normal land production; and an installation that has the appearance and charm of a natural island.

The general conclusion has to be that every problem must be considered on its own and in the light of its particular limitations and requirements. If

the conditions are proper and if the design and construction are done in a modern engineering manner and not ruled too much by traditional methods and outmoded equipment (which have often led to failure of less critical marine construction) a rubble mound, sloping-sided island can be effective, long lasting and economical as well as attractive in appearance.

The Complete Project

Immediately upon assuming essential occupancy of the island and the causeway, Richfield Oil Corporation started its drilling program and the installation of oil field equipment and piping. Several wells have been completed at the time of this writing. When the drilling program is entirely completed, there will be no derrick on the island. The palm trees, installed by the Owner, enhance the natural appearance of the project which is visible to the many people traveling north and south along the coast via highway, train and plane. The causeway as well as the island receives considerable attention since the alternate single and double (battered) pile bents provide a clean appearance and constantly changing line patterns as the viewer travels along the shoreline.

CREDITS

It took the combined effort of many persons to bring the concept of a man-made island and causeway into physical existence. Only a few of these can be mentioned. The Owner of the installation and the responsible party for the island location and oil production facilities is Richfield Oil Corporation. The project was under the production department headed by W. J. Travers vice president, with Karl Kreiger manager of operations, and R. O. Pollard manager of the southern division which will operate the facility. The general contractor for the island proper was Guy F. Atkinson Company with D. E. Root, vice president in charge, Edward Raimer and Charles Thompson, field superintendents for the island construction and quarry operations, respectively. The general contractor for the causeway was Healy Tibbitts Construction Company with R. H. Smith in charge and James Lees as field superintendent. Establissements Neyrpic and Sotramer of Grenoble, France are holders of patents on the tetrapods. The design, wave research, general engineering supervision and inspection were conducted by John A. Blume & Associates, Engineers with the consulting aid of Paul Horrer on wave predictions and Robert Y. Hudson on revetment stability and wave action model testing. The latter operation was conducted at the U. S. Army Engineer's Waterways Experiment Station, Vicksburg under special financial arrangements. H. J. Sexton was engineer in charge of design; James M. Keith was design engineer and later resident engineer at the wave laboratory and in the field during all construction operations; John A. Blume was the firm principal in direct charge of the overall project.

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LABORATORY INVESTIGATION OF RUBBLE-MOUND BREAKWATERS

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ABSTRACT

Laboratory tests have been conducted to determine stability characteristics of various armor-unit shapes, and a new stability formula for rubble breakwaters was developed. Stability, thickness, and porosity data are presented for quarry-stone and tetrapod armor units. Tests of tribars are in progress, and other shapes of armor units will be tested.

SYNOPSIS

A laboratory investigation is being conducted at the U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, to determine criteria for the design and construction of rubble-mound breakwaters. Small-scale breakwater sections are hand-constructed in a concrete wave flume 119 ft long, 5 ft wide, 4 ft deep, and subjected to mechanically generated waves to determine the stability of the armor units.

A general stability equation has been derived and is being used to guide the experimental program and correlate the test data. From the test data obtained to date, important unknown functions in the general stability equation have been determined for selected breakwater and test-wave conditions, and a new breakwater stability formula has been obtained.

In conjunction with the stability tests, wave run-up data are obtained for each breakwater section and wave condition tested. Also, measurements are obtained that enable the thickness and porosity of cover layers composed of different types of armor units to be determined.

The new stability formula and the experimental data obtained so far have provided essential information for an improved method of designing rubble-mound breakwaters with protective cover layers composed of quarry-stone and

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tetrapod armor units. Tests now in progress to obtain experimental data for other special shapes of cast-concrete armor units (cubes, tetrahedrons, and tribars) should increase considerably the accuracy of rubble-mound breakwater design.

INTRODUCTION

Small-scale tests of rubble-mound breakwaters have been in progress at the U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, almost continuously since 1942. During the period 1942-1950, various phases of rubble-mound breakwater construction were investigated for the Bureau of Yards and Docks, Department of the Navy. The most important findings of that investigation concerned the accuracy of Iribarren's formula.(1,2,3) It was concluded(4) that the Iribarren formula can be used for the design of rubble-mound breakwaters only if experimental coefficients, or the kind developed during the investigation conducted for the Bureau of Yards and Docks, are available for the complete range of variables encountered in the design of full-scale structures.

In 1951 a comprehensive investigation of rubble-mound breakwaters(5) was begun at the Waterways Experiment Station for the Office, Chief of Engineers, U. S. Army. This investigation, which is still in progress, is similar to the study conducted for the Bureau of Yards and Docks except that it is larger in scope, including the necessary range of important variables that affect the stability of rubble-mound breakwaters.

To insure optimum designs for breakwaters, design engineers should have accurate information concerning the required weight for the individual armor units in the protective cover layer, along the length of the structure, as a function of: (a) shape of unit, (b) specific weight of unit, (c) specific weight of water in which the structure will be situated, (d) beach slope seaward of the breakwater, (e) dimensions of waves at the location of the proposed structure, (f) seaside slope of breakwater, (g) porosity of protective cover layer, (h) thickness of cover layer, and (i) porosity and thickness of underlayers upon which the armor units are to be placed. In addition, design engineers should be able to determine quantitatively: (a) the height of breakwater above still-water level necessary to prevent excessive overtopping by wave run-up, (b) the depths below still-water level to which the cover layer should extend, (c) the amount of damage that will be inflicted on a breakwater section not designed for overtopping when waves higher than the selected design wave occur, and (d) the best design of back slopes for preventing failure when overtopping of the breakwater is permitted. Information should also be available for designing the seaward end, or head, of the breakwater.

The present test program includes tests to provide the design data and quantitative information outlined in the preceding paragraph. However, tests completed to date, and described in this paper, have been concerned for the most part with only two types of rubble-mound breakwaters: one in which that part of the breakwater section subjected to the most intense wave action is composed of a pile of quarry-stone armor units placed pell-mell; and the other in which the protective cover layers are composed of two layers of cast-concrete armor units placed pell-mell over one or two quarry-stone underlayers.

After the comprehensive investigation was begun, it was found that the Iribarren formula has limitations that render it unsatisfactory for use in correlating stability data from tests of small-scale rubble-mound breakwaters. Thus it was necessary to reanalyze the phenomenon that results when waves attack a rubble-mound breakwater in order to develop a more general stability equation.

This paper describes the apparatus and testing techniques being used in the laboratory investigation, explains why it was considered necessary to abandon the use of Iribarren's formula in correlating test data, and presents the derivation of a more general stability equation which, with the experimental data obtained to date, was used to develop a simple formula for the weight of armor units necessary to insure the stability of rubble-mound breakwaters. This paper also presents information concerning wave run-up, and the thickness and porosity of cover layer materials.

For this paper, a rubble-mound breakwater is considered to be one constructed with a core of quarry-run stones, sand, slag, or other suitable materials, protected from wave action by one or more stone underlayers and a cover layer of relatively large, selected quarry stones or specially-shaped concrete armor units.

Discussion of Iribarren's Formula

Iribarren's original formula for the weight of armor units in rubble-mound breakwaters, in its general form, revised(6) to make it dimensionally homogeneous, and retaining the coefficient of friction as a variable, reduces to

$$W_r = \frac{K' \gamma_r \mu^3 H^3}{(\mu \cos \alpha - \sin \alpha)^3 (S_r - 1)^3} \qquad (1)$$

where W_r is the weight of individual armor units, γ_r is the specific weight of the armor units, S_r is the specific gravity of the armor units relative to the water in which the breakwater is situated ($S_r = \gamma_r/\gamma_w$), μ is the effective coefficient of friction between armor units, H is the height of wave attacking the breakwater, α is the angle, measured from the horizontal, of the exposed breakwater slope, and K' is an experimentally determined coefficient. The accuracy of this formula was discussed by Hudson and Jackson,(4) and Hudson(6) in 1953. At that time it was concluded that the Iribarren formula could be used to correlate the test data, and that it could be made sufficiently accurate for use in designing full-scale rubble-mound breakwaters, if sufficient test data were available to evaluate the experimental coefficient (K').

After the comprehensive testing program was begun, and shortly after the conclusions concerning the adequacy of Iribarren's formula were published, preparations were initiated for tests to determine the stability of armor units as a function of armor-unit shape. These included a study to establish the values of the friction coefficient (μ) that should be used for the various shapes of armor units in the experimental determination of K' in Iribarren's formula. The first armor units of special shape for which friction coefficients were measured were cubes and tetrapods. Tetrapod is the name of a patented armor unit of special shape that was developed at the Laboratoire Dauphinois d' Hydraulique Ets. Neyrpic, Grenoble, France.(7) The tests showed that the friction coefficient in Iribarren's formula, as measured by the tangent of the angle of repose (ϕ), varied appreciably with the shape of armor unit and the

method of placing these units in the cover layer. These results led to the realization that the experimental coefficient (K') in Iribarren's formula could not be determined accurately from small-scale breakwater stability tests unless accurate comparative values of the friction coefficient could be obtained for the different shapes of armor units. This realization was made more acute by the fact that Iribarren's force diagram, from which his basic stability equation was derived, is predicated on the assumption that the friction between armor units, specifically that component of the friction force parallel to the breakwater slope, is the primary force that resists the forces of wave action and determines the stability of the armor units.

Results of coefficient-of-friction determinations for three sizes of quarry stones, and for concrete cubes and tetrapods are shown in Table 1. Fig. 1 shows the shapes of these armor units. About seventy repeat tests of the 0.30-lb, quarry-stone armor units were conducted to determine the range of μ for units of this type. It was found that μ varied from a low of 0.78 to a high of 1.28, with an average value of 0.98. Thus, μ varies not only with armor-unit shape and method of placing, but it also varies considerably from test to test for the same armor unit. The curves of Fig. 2 were prepared using the modified Iribarren formula (Eq. (1)), and show the effects of variations in the measured value of μ on the calculated values of K'. Since W_r is directly proportional to K', variations in μ have the same effect on calculated values of W_r as they do on K'. It can be seen that for steep breakwater slopes, small variations in the measured value of μ cause large variations in the calculated values of K' and W_r. This becomes more significant when it is recalled that the use of concrete armor units of special shape is more apt to be economically feasible only for the steeper breakwater slopes.

Based on the results of the tests to determine friction coefficients, correlation of test data by the use of Iribarren's formula was abandoned, and a new stability equation, similar to the Iribarren formula but capable of more general application, was derived.

Table 1

Friction Coefficients ($\mu = \tan\phi$) of Armor Units

Method of Measurement	Quarry Stone W_r = 0.10 lb	Quarry Stone W_r = 0.30 lb	Quarry Stone W_r = 0.62 lb	Concrete Cubes W_r = 0.80 lb	Concrete Tetrapods W_r = 0.21 lb
① Dumped in water	1.02	0.98	1.13	1.20	1.10
② Dumped in air	0.79	0.90	0.87	1.34	---
③ Stacked in water	1.09	1.19	1.26	1.36	1.78
④ Stacked in air	0.97	1.12	1.22	1.75	---
Avg (all methods)	0.97	1.05	1.12	1.41	---
Avg (① and ③)	1.06	1.09	1.20	1.28	1.44

Analytical Bases of Stability Equation

When short-period wind waves impinge upon a pervious rubble-mound breakwater, the resulting interplay of forces developed by the wave-induced water motion and the resisting action of the armor units in the cover layer is extremely complex, and attempts to describe the phenomenon quantitatively by rigorous theoretical analyses have not, as yet, been successful. Waves at a breakwater may break completely, projecting a jet of water approximately perpendicular to the slope, break partially with a poorly defined jet, or establish an oscillatory motion of the water particles along the breakwater slope similar to the motion of a clapotis at a vertical wall. Characteristics of the motion of water particles when short-period wind waves encounter a rubble-mound breakwater are determined by the wave steepness (H/λ), the relative depth (d/λ), the relative height (H/d), the depth of water at the toe of the breakwater slope (d), the angle of the beach slope seaward of the breakwater (σ), angle of seaside slope of the breakwater with the horizontal (α), the angle of obliquity of the attacking waves (β), and the shape, thickness, and porosity of the cover layer and underlayer materials (Δ, r, and P, respectively).

The ability of an armor unit in the cover layer to resist the forces caused by wave action is determined by: the buoyant weight of the armor unit (W_r'), position of the unit relative to the still-water level (z), the angle of seaside slope (α), the height of breakwater crown above still-water level (h), the width of breakwater crown (m), the shape of unit (Δ), porosity of the armor units in place (P), thickness of the cover layer (r), the porosities and thicknesses of the underlayers, and the method of placing the breakwater material, especially the armor units in the cover layer (dumped pell-mell, placed in some orderly manner to obtain wedging action, or stacked without wedging action).

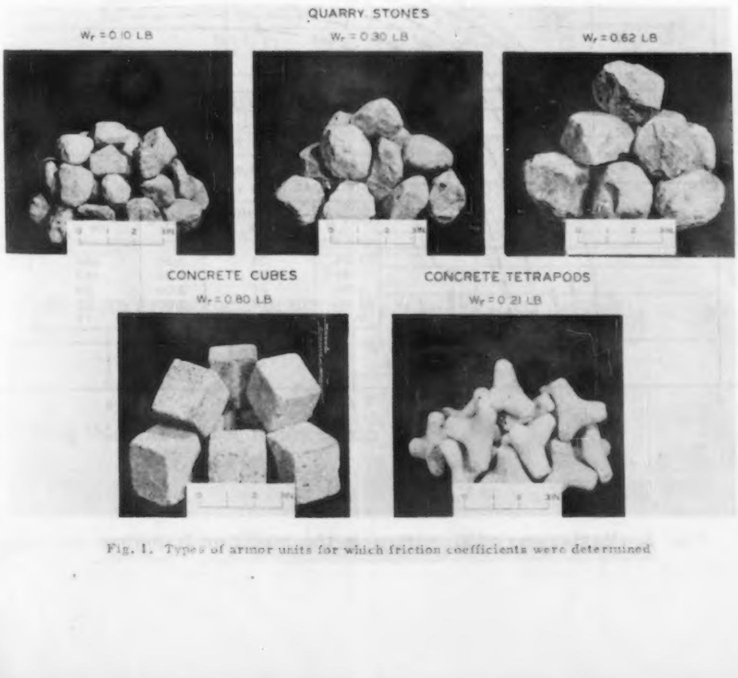

Fig. 1. Types of armor units for which friction coefficients were determined

Short-period wind waves incident upon a rubble-mound breakwater develop dynamic forces that tend to lift and roll the armor units from the breakwater slope. These forces consist of a drag force

$$F_d = \frac{1}{2} C_d k_a \ell^2 \frac{\gamma_w}{g} V^2 \qquad (2)$$

and an inertia force

$$F_m = C_m k_v \ell^3 \frac{\gamma_w}{g} \frac{\partial V}{\partial t} \qquad (3)$$

where C_d is a drag coefficient, C_m is a virtual-mass coefficient, ℓ is a characteristic linear dimension of the unit such that the projected area of the unit perpendicular to the velocity is $k_a \ell^2$ and the volume of the unit is $k_v \ell^3$, γ_w is the specific weight of the water in which the breakwater is to be situated, g is acceleration due to gravity, and V is the velocity of the water flowing around or impinging on the armor units in the cover layer. Because of the difficulties inherent in an attempt to evaluate the separate sets of coefficients $C_d k_a$ and $C_m k_v$, which would involve either direct measurement or a derived expression of the acceleration $(\partial V/\partial t)$ in terms of the wave characteristics, and in order to simplify the force equation used to correlate test data, the effects of acceleration are combined with the drag force. The resulting equation is

$$F_q = C_q \ell^2 \frac{\gamma_w}{g} V^2 \qquad (4)$$

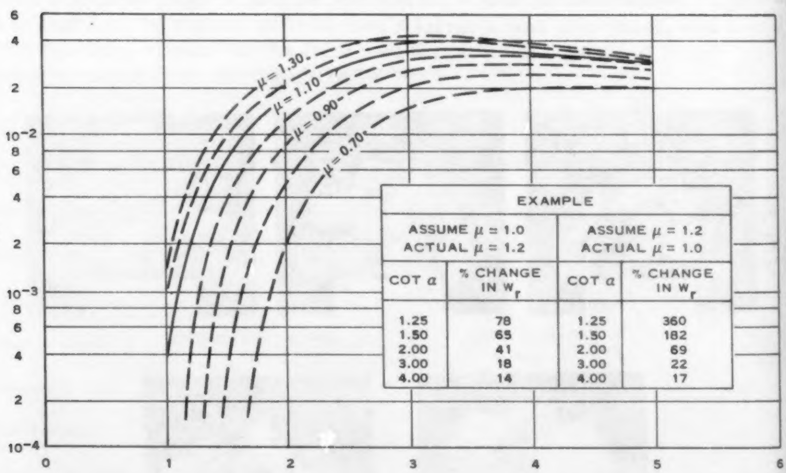

Fig. 2. Variations of K' with μ in the modified Iribarren formula

where C_q, the total coefficient, is a function of the terms $\frac{\ell}{V^2}\frac{\partial V}{\partial t}$, $C_d k_a$, and $C_m k_v$.

The velocity of the water jet resulting from a breaking wave (V_b) is equal to the particle velocity at the wave crest which, at the instant of breaking, is equal to the celerity of the wave form. Thus, for shallow-water waves, as $d/\lambda \longrightarrow 0$,

$$V_b^{\,2} = g d_b \tag{5}$$

Also at breaking, $H_b = k d_b$ where $k = f(H/\lambda)$. Therefore, by substitution,

$$V_b^{\,2} = \frac{g}{k} H_b \tag{6}$$

Substituting this value of velocity in Eq. (4), the expression for the force exerted on an armor unit by a breaking wave, in terms of wave height, is

$$F_q = C_q \ell^2 \frac{\gamma_w}{k} H_b \tag{7}$$

For breakwaters constructed by dumping or by placing armor units essentially pell-mell, the forces resisting displacement are the buoyant weight of the individual units and the friction between units. Except for isolated instances where wedging action is involved, friction between armor units can be neglected, and the principal resisting force for pell-mell-constructed cover layers can be assumed to be

$$W'_r = k_v \ell^3 (\gamma_r - \gamma_w) \tag{8}$$

where γ_r is the specific weight of the armor units.

For incipient instability of armor units in a rubble-mound breakwater, or fill slope, subjected to breaking waves, $W'_r = F_q$, or

$$k_v \ell^3 (\gamma_r - \gamma_w) = C_q \ell^2 \frac{\gamma_w}{k} H_b \tag{9}$$

Letting $S_r = \gamma_r/\gamma_w$, and substituting in Eq. (9),

$$k_v \ell (S_r - 1) = \frac{C_q H_b}{k}$$

or

$$\frac{H_b}{\ell (S_r - 1)} = \frac{k(k_v)}{C_q} \tag{10}$$

The weight of an armor unit in air is $W_r = k_v \ell^3 \gamma_r$, or

$$\ell = \left(\frac{W_r}{k_v \gamma_r}\right)^{1/3} \tag{11}$$

Substituting this value of ℓ in Eq. (10),

$$\frac{\gamma_r^{\,1/3} H_b}{(S_r - 1) W_r^{\,1/3}} = \frac{k(k_v)^{2/3}}{C_q} \tag{12}$$

where

$$\frac{k(k_v)^{2/3}}{C_q} = f\left(C_d, k_a, C_m, k_v, \frac{\ell}{V^2}\frac{\partial V}{\partial t}, d/\lambda, H/\lambda\right)$$

The forces that tend to displace armor units from breakwater slopes when the waves do not break, or break only partially, are not the same as those forces that result from breaking waves, nor do they act in the same direction. However, the order of magnitude of the nonbreaking wave forces, and the effects of these forces on the stability of rubble-mound breakwaters, should be approximately the same as those caused by breaking waves. It is believed, therefore, that Eq. (12) adequately represents, at least in the first approximation, the major forces of both breaking and nonbreaking waves. Thus, for both types of short-period wave motions on rubble-mound breakwaters, and introducing those variables that were not included in the derivation of Eq. (12), the most general equation used in this investigation to guide the testing program and correlate test data is

$$\frac{\gamma_r^{1/3} H}{(S_r - 1) W_r^{1/3}} = f\left(\begin{array}{l}\alpha, C_d, C_m, k_a, k_v, \frac{\ell}{V^2}\frac{\partial V}{\partial t}, H/\lambda, d/\lambda, \\ H/d, d, \sigma, P, r, h, m, z, \beta, \text{and} \\ \text{the method of placing armor units}\end{array}\right) \qquad (13)$$

In Eq. (13), C_d and C_m are functions of Δ and the Reynolds number (R), and k_a and k_v are functions of Δ. The term $\frac{\ell}{V^2}\frac{\partial V}{\partial t}$, which is a form of Iversen's modulus for accelerated motion,(8) is omitted from the list of variables tested in this investigation because of the difficulty of obtaining accurate velocity-time histories of the flow around individual armor units.

In the first phase of the present testing program, the upper portion of the small-scale breakwaters was constructed of rocks simulating quarry stones, all pieces of which were of nearly the same weight, specific weight, and shape. In addition, the crown width of the breakwater test sections was standardized at three times the average diameter of the armor units, the angle of obliquity of the test waves was 0 deg, and the cover layer was extended to a depth below still-water level sufficient to insure that the stability of the structure would not be influenced by the stones used in the lower portion of the test section. For those tests in which the no-damage criterion was used in the selection of design-wave heights, the crown heights above still-water level were sufficient to prevent overtopping by the test waves. For those tests in which the wave heights used were greater than the previously selected design-wave heights, the crown heights above still-water level, and the depths to which the cover layers extended below still-water level, were equal to the previously selected design-wave heights. For all tests, the water depth between the wave generator and the breakwater was constant, and was sufficient to prevent the ratio H/d from influencing the action of waves on the structure. For the tests conducted, the variation in Reynolds number was comparatively small. Tests in a larger wave flume at the laboratory of the Beach Erosion Board, CE, Washington, D. C., are being conducted to determine the effects of this variable on the stability of armor units in rubble-mound breakwaters.

When damage is allowed to occur to the breakwater (by use of wave heights greater than the design-wave height), the geometry of the structure, the moti

of the water particles, and the resulting forces on the breakwater differ from those resulting from tests in which the no-damage criterion is used. Thus a damage parameter, D, defined as the percentage of armor units displaced from the cover layer by wave action, is included as a prime variable.

For the breakwater sections investigated in the first phase of the testing program, in which the armor units were rocks simulating rounded and smooth quarry stones placed pell-mell

$$\frac{\gamma_r^{1/3} H}{(S_r - 1) W_r^{1/3}} = f(\alpha, H/\lambda, d/\lambda, \text{and } D) \tag{14}$$

In the second phase of the testing program the armor units used were patterned after the tetrapod, and the rubble mound was protected by two or more layers of armor units placed over one or two quarry-stone underlayers. For these tests

$$\frac{\gamma_r^{1/3} H}{(S_r - 1) W_r^{1/3}} = f(\alpha, H/\lambda, d/\lambda, r) \tag{15}$$

The dimensionless parameter on the left side of Eqs. (13) - (15) is designated the stability number (N_s) for rubble-mound breakwaters.

Experimental Equipment and Procedure

Test Apparatus

The breakwater stability tests are conducted in a concrete flume 5 ft wide, 4 ft deep, and 119 ft long, equipped with a plunger-type wave generator. Wave heights are measured with a parallel-rod-type wave gage, and recorded on a direct-writing oscillograph. The wave-height measuring apparatus consists of the wave gage (two 1/8-in. stainless steel, parallel rods 1.2 ft long, spaced 2 in. apart), a balancing circuit, a Brush universal analyzer (Model No. BL-320), and a Brush magnetic oscillograph (Model No. BL-201).

Cross-section measurements of the small-scale breakwaters are obtained with a sounding rod equipped with a circular spirit level for plumbing, a scale graduated in thousandths of a foot, and a ball-and-socket foot which facilitates adjustment to the irregular surface of the breakwaters. The foot is circular, and for each test the diameter of the foot is equal to one-half the average diameter of the armor units.

Types of Tests Conducted

Two primary types of stability tests are being conducted in this investigation. First, design-wave heights are determined for breakwater sections of sufficient height to prevent overtopping by the test waves. Design-wave height is defined as the maximum wave height, measured at the location of a proposed breakwater before it is constructed, which will not damage the cover layer. The removal of up to one per cent of the total number of armor units in the cover layer is considered to be "no damage."

The second type of tests being conducted is concerned with determination of safety factors for breakwater sections designed on the basis of the criteria

established from results of the no-damage and no-overtopping tests. For the safety-factor tests, breakwater sections are constructed in the wave flume in accordance with the results of the no-damage and no-overtopping tests, and the amount of damage, as determined by the percentage of armor units removed from the cover layer, is obtained as a function of wave height. Wave heights greater than the previously selected design-wave height for the no-damage and no-overtopping criteria are used in these tests.

In addition to the two above-mentioned types of tests, special tests are conducted from time to time to determine optimum designs for specific breakwaters. In these tests, design-wave heights may be determined for conditions other than no-damage and no-overtopping.

Breakwater Sections Tested

Rubble-mound breakwaters of the types shown schematically in Figs. 3 and 4 have been used in most of the stability tests. In the no-damage and no-overtopping tests, the crown heights were sufficient to prevent overtopping, and the cover layer was extended a sufficient distance below still-water level to prevent damage to the class B stones used below the armor units. The distance below still-water level to which the armor units extended, as well as the height of the breakwater crown above still-water level, was equal to or greater than the wave heights used to test the breakwater sections. In the safety-factor tests of quarry-stone armor units, the crown heights above still-water level, and the maximum distances below still-water level to which the armor units in the cover layers extended, were numerically equal to the design-wave heights previously selected in the corresponding no-damage and no-overtopping tests.

In a few tests to determine the stability of the Crescent City Harbor breakwater,(9) the type breakwater section shown in Fig. 5 was used. This breakwater section was designed for overtopping.

Types of Breakwater Materials Used

Quarry-Stone Armor Units and Class B Stones.—In each stability test the quarry-stone armor units were as nearly the same weight, specific weight, and shape as possible. Both the armor stones and class B stones were sized from crushed basalt. The weights of class B stones were approximately the same as those of the armor stones; however, the class B stones were sized by means of sieves, whereas each armor stone was sized and shaped by hand and weighed on a torsion balance having a sensitivity of one-tenth gram. Two sizes of armor stones were used to insure that the design-wave heights, and the heights of waves used in the safety-factor tests, would be within the range of wave dimentions that the wave machine can generate. Approximately 2800 pieces of the larger-size armor stones were used. Based on a representative sample of 175 pieces, the average weight and specific weight of the larger-size armor stones were 0.30 lb and 176.0 lb per cu ft, respectively. Based on a representative samples of 475 pieces, the average weight and specific weight of the smaller-size armor stones were 0.10 lb and 174.7 lb per cu ft, respectively. The core material, which was the same for all tests conducted, consisted of crushed basalt with a mean particle diameter of 1/8 in.

Tetrapod Armor Units.—Tests have been conducted using tetrapod-shaped armor units molded of both concrete and leadite. Leadite is the trade name for a caulking compound which has a specific weight nearly the same as that

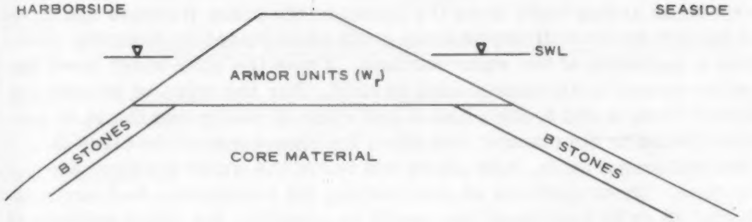

Fig. 3

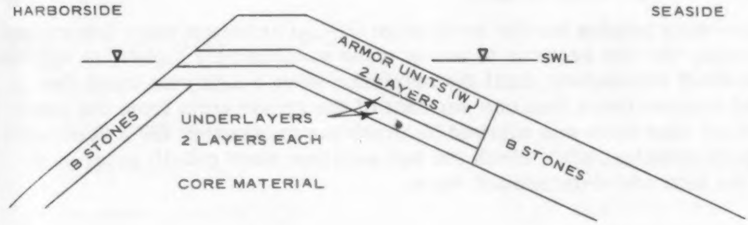

Fig. 4

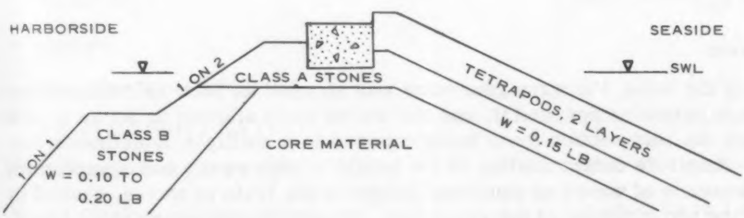

Fig. 5

Figs. 3, 4, and 5. Elements of breakwater sections tested

of the concrete used to mold the tetrapod armor units. Based on representative samples of 125 pieces, the average weight and specific weight of the concrete tetrapods were 0.21 lb and 142.3 lb per cu ft, respectively, and the average weight and specific weight of the leadite tetrapods were 0.22 lb and 140.4 lb per cu ft, respectively.

Method of Constructing Test Sections

The breakwater test sections were constructed in the wave flume on a sand base 85 ft from the wave generator. The core material and class B stones from the base of the test section to the crown of the core were placed with the flume dewatered. The core material was wetted with a hose and then compacted with hand trowels to simulate the natural consolidation effected by wave action during construction of full-scale structures. The class B stones were then placed by shovel and dressed by hand, after which the flume was flooded

to the desired still-water level. For the type of breakwater shown in Fig. 3, the quarry-stone armor units from the crown of the class B stones and core-material section to the still-water level (swl) were placed by dumping, pell-mell, from a container at the water surface. Above the still-water level the quarry-stone armor units were placed by hand. For the types of breakwater illustrated in Figs. 4 and 5, the class A and class B stones and the core material were placed in the manner described for placement of the class B stones, and the armor units, both above and below the water surface, were placed by hand. These methods of constructing the breakwater test sections were adopted so as to reproduce, as nearly as possible, the usual methods of constructing full-scale structures.

Selection of Design-Wave Heights

Design-wave heights for the no-damage ($H_{D=0}$) criterion were determined by subjecting the test sections to waves made successively higher, in approximately 0.02-ft increments, until the maximum wave height was found that would not remove more than one per cent of the armor units from the cover layer. Each size wave was allowed to attack the breakwater for a cumulative period of 30 minutes, after which the test sections were rebuilt prior to attack by the next added-increment wave.

Range of Test Conditions

The tests involved the ranges of wave and breakwater characteristics listed in Table 2.

Test Waves

During the tests, the wave generator was stopped as soon as reflected waves from the breakwater reached it, and the waves were allowed to decay in order to prevent the test section from being exposed to a multiple, undefined wave system. Accurate determination of the height of test waves was complicated by the presence of waves of abnormal height in the train of waves, caused by the starting and stopping of the generator. Usually there were one or two large waves at the end of each cycle. The larger waves, which occurred about one per cent of the time that waves attacked the test structure, averaged about 12 per cent higher than the average height of the highest one-third of the waves in the wave trains ($H_{1/3}$). Waves of height $H_{1/3}$ are called the "significant" waves of fully established wave trains in nature. It has been determined(10) that storm-wave trains in nature contain waves about 25 per cent larger than the significant wave 5 per cent of the time, 33 per cent larger 3 per cent of the time, and 58 per cent larger 1 per cent of the time. The importance of these facts with respect to the design of rubble-mound breakwaters is not fully understood at the present time. However, it is believed that the existence of these larger-size waves in natural wave trains must be considered in the selection of design-wave heights and factors of safety.

$$N_s = a\ (\cot \alpha)^{1/3} \tag{16}$$

or

$$\frac{\gamma_r^{1/3} H_{D=0}}{W_r^{1/3}(S_r - 1)} = a\ (\cot \alpha)^{1/3} \tag{17}$$

Table 2

Ranges of Wave and Breakwater Characteristics Tested

Characteristic	Range of Test Conditions
Wave height (H)	0.28 to 0.69 ft
Water depth (d)	1.26 and 2.00 ft
Wave period (T)	0.88 to 2.65 sec
Wave length (λ)	4.0 to 20.0 ft
Relative depth (d/λ)	0.10 to 0.50
Wave steepness (H/λ)	0.015 to 0.128
Specific weight of:	
Quarry stones (γ_r)	166.0 to 191.6 lb per cu ft
Concrete tetrapods (γ_r)	135.0 to 154.0 lb per cu ft
Leadite tetrapods (γ_r)	134.0 to 142.0 lb per cu ft
Water (γ_w)	62.4 lb per cu ft
Weight of:	
Quarry stones (W_r)	0.09 to 0.31 lb
Concrete tetrapods (W_r)	0.18 to 0.24 lb
Leadite tetrapods (W_r)	0.21 to 0.23 lb
Breakwater slope ($\tan \alpha$)	1 on 1.25 to 1 on 5

from which, if $K_\Delta = a^3$,

$$W_r = \frac{\gamma_r \left(H_{D=0}\right)^3}{K_\Delta (S_r - 1)^3 \cot \alpha} \tag{18}$$

This is the desired stability formula for quarry-stone and tetrapod-shaped armor units for the no-damage and no-overtopping conditions. The test data indicate that, for pell-mell placing of armor units, the experimentally determined coefficient (K_Δ) varies primarily with the shape of the armor units. The values of K_Δ for quarry-stone and tetrapod-shaped armor units, corresponding to the best-fit lines AB and MN of Fig. 6, are 3.2 and 9.5, respectively.

Tests conducted previously showed that, for the type of breakwater tested and for breakwater slopes flatter than 1 on 2, the stability number increases slightly as the number of layers of armor units is increased from 2 to 4. Although an increase in stability number means a decrease in weight of armor unit for the same wave height, the saving in volume of material per armor unit

is more than offset by the increased thickness of the cover layer. These tests indicated that n = 2 is the optimum for tetrapod cover layers.

Damage or Safety-Factor Tests

Because storm-wave trains contain waves higher than the significant height ($H_{1/3}$), it is important that rubble-mound breakwaters be designed so that they will not fail when subjected to waves with heights moderately larger than the selected design-wave height. Thus quarry-stone armor units were subjected to tests in which wave heights were greater than the previously selected design-wave heights for the no-damage and no-overtopping criteria to obtain information concerning safety factors for rubble-mound breakwaters designed on the basis of Eq. (18). Results of these tests for quarry-stone armor units are presented in the form of a log-log plot in Fig. 7, with the stability number (N_s) as the ordinate, cot α as the abscissa, and the percentage of damage to the cover layer (D) as the parameter. The damage tests were conducted using 0.10-lb and 0.30-lb armor stones and relative depths of 0.10 and 0.25. The solid line AB in Fig. 7 is the same as line AB in Fig. 6, i.e., it is the approximate best-fit line through the data points for the no-damage and no-overtopping criteria. The dashed lines in Fig. 7 were drawn parallel to line AB through data points delineating approximate ranges of percentages of damage to the

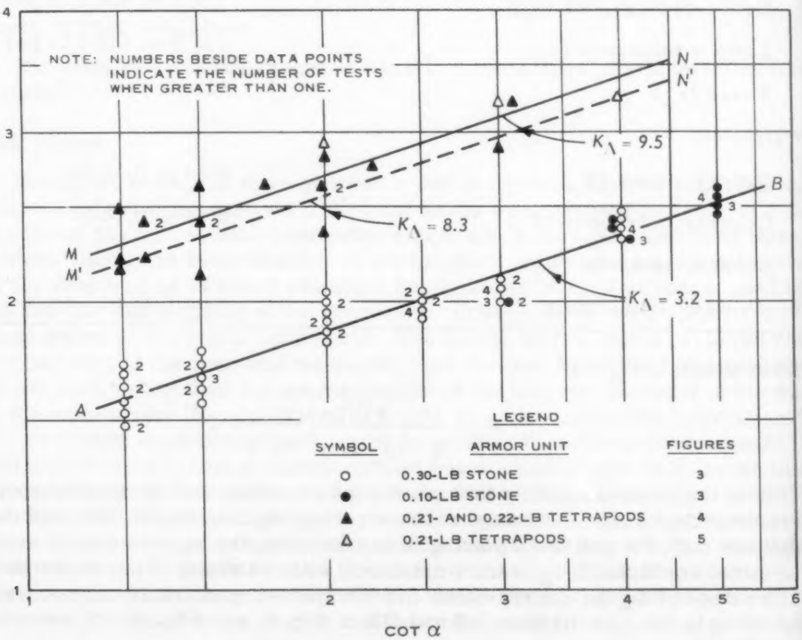

Fig. 6. Stability of quarry-stone and tetrapod armor units: N_s as a function of Δ and α for the no-damage and no-overtopping criteria

cover layer. Although the dashed lines represent only rough approximations of the amounts of damage obtained for the different wave heights, it is believed that, considering the nature of the tests and the significance of the damage parameter, they reflect the test results with sufficient accuracy for the immediate needs of the design engineer.

The form of the equation for the dashed lines in Fig. 7 is the same as that of lines AB and MN of Fig. 6; therefore, the general formula for stability of quarry-stone armor units, for $H \geqq H_{D=0}$, is

$$W_r = \frac{\gamma_r H^3}{K_D (S_r - 1)^3 \cot \alpha} \tag{19}$$

where K_D is the experimentally determined damage coefficient, and H is the corresponding wave height. Table 3 shows values of D, $H/H_{D=0}$, and K_D corresponding to the various lines in Fig. 7.

In this tabulation, the amounts of damage to the test sections are given in terms of percentages of the armor units removed from the cover layer. In the

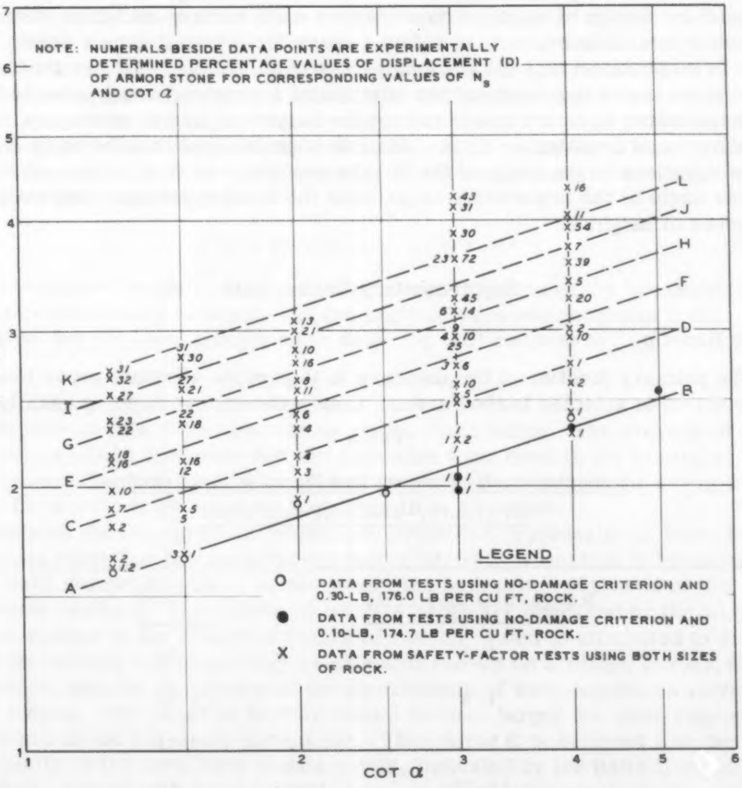

Fig. 7. Stability of quarry-stone cover layers: N_s as a function of α and D

damage tests the breakwater sections were of the type shown in Fig. 3, with both the crown height above still-water level and the maximum distance below still-water level to which the armor units extended equal in magnitude to the previously determined design-wave height. Thus, the percentages of damage for these tests are considerably smaller than the corresponding percentages of damage which would obtain for breakwaters of the type shown in Fig. 4, other conditions being equal. In the Fig. 4 breakwater section, the volume of the cover layer is smaller than that shown in Fig. 3; consequently, for equal amounts of damage to the cover layer, the percentage of damage is proportionally larger for the cover layer of smaller volume.

Comprehensive tests to determine the amount of damage to tetrapod cover layers as a function of $H/H_{D=0}$ have not been conducted. However, preliminary tests of tetrapods in which waves larger than $H_{D=0}$ were used indicate that the limit of stability of tetrapod armor units, with n = 2, is reached when the ratio $H/H_{D=0}$ becomes equal to approximately 1.2. For values of $H/H_{D=0}$ slightly larger than 1.3, failure of the tetrapod cover layer occurs. It is believed, therefore, that a value of K_Δ of 8.3, which corresponds approximately to the lower envelope of data points for tetrapods in Fig. 6, line M'N', should be used for design of tetrapod cover layers until more quantitative information is available concerning safety factors for tetrapod armor units.

It is emphasized that the wave heights in Eqs. (18) and (19) are the selected significant waves that occur at the position of a proposed breakwater before the breakwater is constructed, and not the heights of waves moving up, or breaking on, a breakwater slope. Also, it is pointed out that the angle (α) in these equations is the angle of the breakwater slope as first constructed, and not the angle of the breakwater slope after the breakwater has been stabilized by waves of height H.

Supplementary Design Data

Wave Run-Up

The primary function of breakwaters is to provide adequate protection from wave action in selected harbor areas. Consequently, overtopping usually can

Table 3

Experimentally Determined Damage Coefficients for Quarry-stone Armor Units

Line	Range of D (per cent)	$H/H_{D=0}$	K_D
AB	0-1	1.00	3.2
CD	1-5	1.18	5.1
EF	5-15	1.33	7.2
GH	10-20	1.45	9.5
IJ	15-40	1.60	12.8
KL	30-60	1.72	15.9

be tolerated only if it is negligible or does not exceed allowable limits as determined by the type of harbor and the use for which different areas in the harbor are designed. There is considerable experimental data in the literature concerning wave run-up on paved slopes, beach slopes, and shore-line structures such as seawalls,(11,12,13,14) and a theoretical method of computing run-up on smooth, impervious slopes by Miche,(15) has been noted in a paper by Bruun.(16) However, comparatively little run-up data are available for structures with slopes as rough and porous as rubble-mound breakwaters.

Although limited in scope, the small-scale tests of wave run-up on sloping structures conducted by Granthem(17) provide some information on this subject. Granthem's tests were conducted in a manner that approximated the action of waves on rubble-mound breakwaters. Although derivation of a theoretical basis for interpretation and correlation of test data was not attempted, it is believed that the important parameters suggested by Granthem's tests can be used to correlate data obtain in the present testing program. Granthem concluded from the results of his tests that the primary variables affecting wave run-up are: the wave steepness (H/λ), relative depth (d/λ), angle of the seaside slope (α), and porosity of the structure (P).

Hydraulic roughness of the slope surface and the angle of obliquity of wave attack (β) are also believed to affect wave run-up. The hydraulic roughness of a breakwater slope is difficult to define quantitatively; however, for the quarry-stone armor units placed pell-mell, such as those used in the investigation discussed in this paper, the average thickness of one layer of armor units should provide an approximate measure of this variable. Thus, correlation of the run-up data for rubble-mound breakwaters may be accomplished by the functional relationship

$$R/H = f(\alpha, H/\lambda, P, d/\lambda, r, \beta) \tag{20}$$

The percentage of voids in the quarry-stone cover layers of the breakwaters tested was essentially constant, and the angle of wave obliquity was 0 deg. Therefore, for the tests completed to date, Eq. (20) reduces to

$$R/H = f(\alpha, H/\lambda, d/\lambda, r) \tag{21}$$

Wave run-up data were obtained by visual observation. The average of five individual readings was recorded for each size wave used in the testing of each section. Each of the five individual readings represented the average run-up for a wave train consisting of from 10 to 15 waves.

Results of the run-up observations are presented graphically in Figs. 8-10. These data show that the wave run-up factor (R/H) is a function of breakwater slope, wave steepness and, to some extent, the hydraulic roughness of the breakwater surface. The effects of relative depth are obscured by the wide range of scatter in the observed values of run-up, which is attributed to difficulties in defining and observing the extent of run-up on a rough, porous, sloping surface, and the complexity of the phenomenon of wave motion on rubble-mound slopes. The range of scatter should be even larger for wave run-up measurements on full-scale structures. Therefore, it is believed that the upper limits of the envelopes of data points, indicated by the solid lines in Figs. 8-10, should be used in selecting design crown elevations when overtopping of a proposed rubble-mound breakwater cannot be tolerated.

The test data show that breakwater slope and wave steepness are primary variables affecting wave run-up on porous rubble-mound breakwaters of the

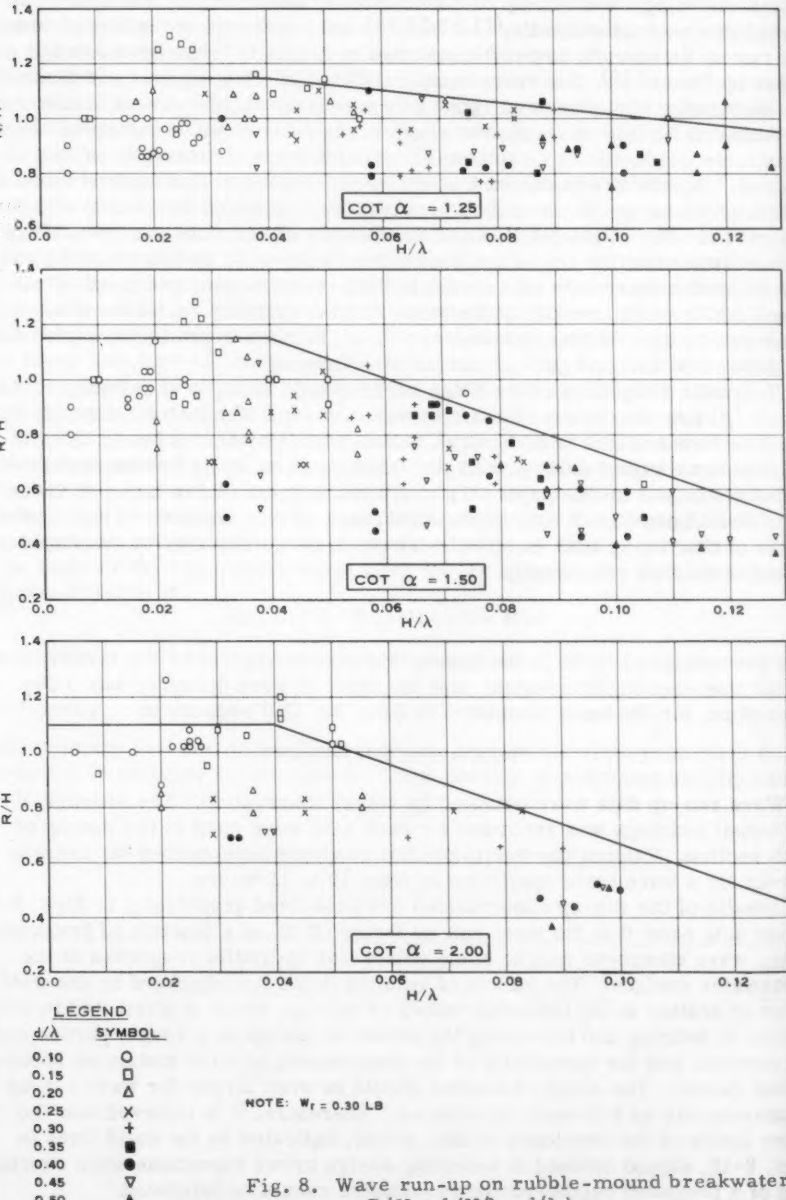

Fig. 8. Wave run-up on rubble-mound breakwaters:
$R/H = f\ (H/\lambda\ ,\ d/\lambda\ )$
$\cot\alpha = 1.25,\ 1.50$, and 2.00

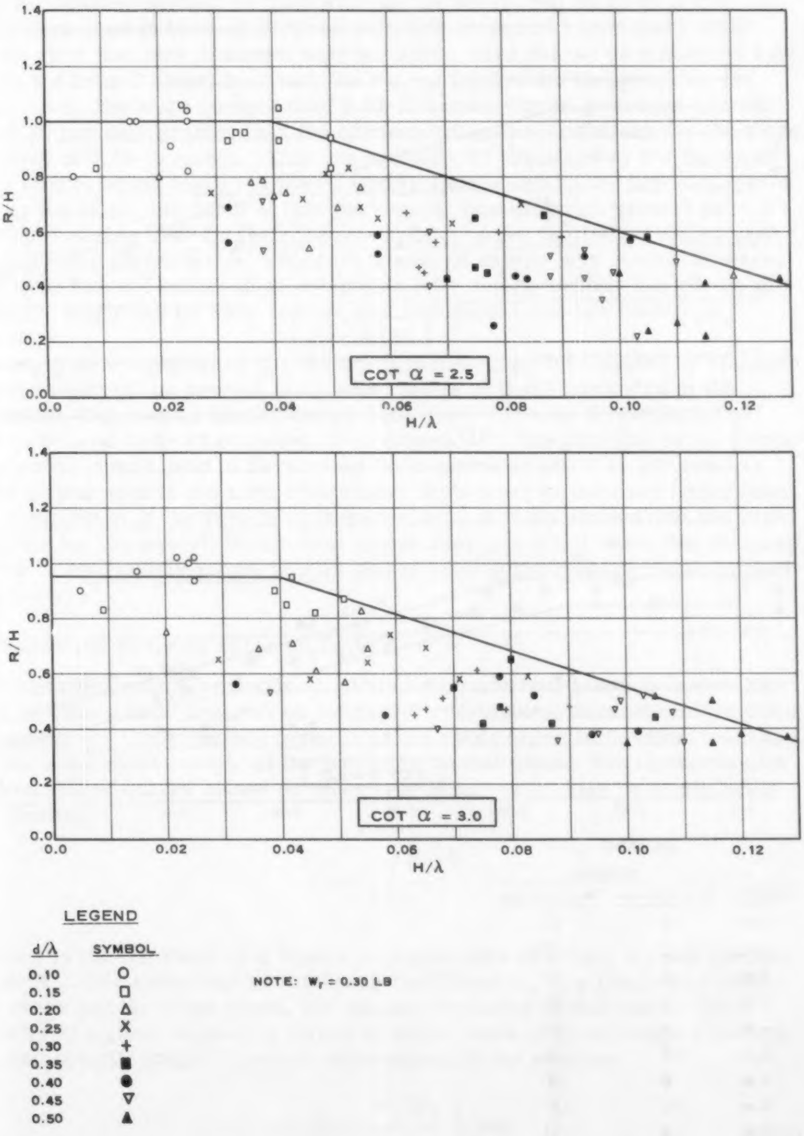

Fig. 9. Wave run-up on rubble-mound breakwaters:
$R/H = f(H/\lambda, d/\lambda)$
$\text{Cot}\,\alpha = 2.5$ and 3.0

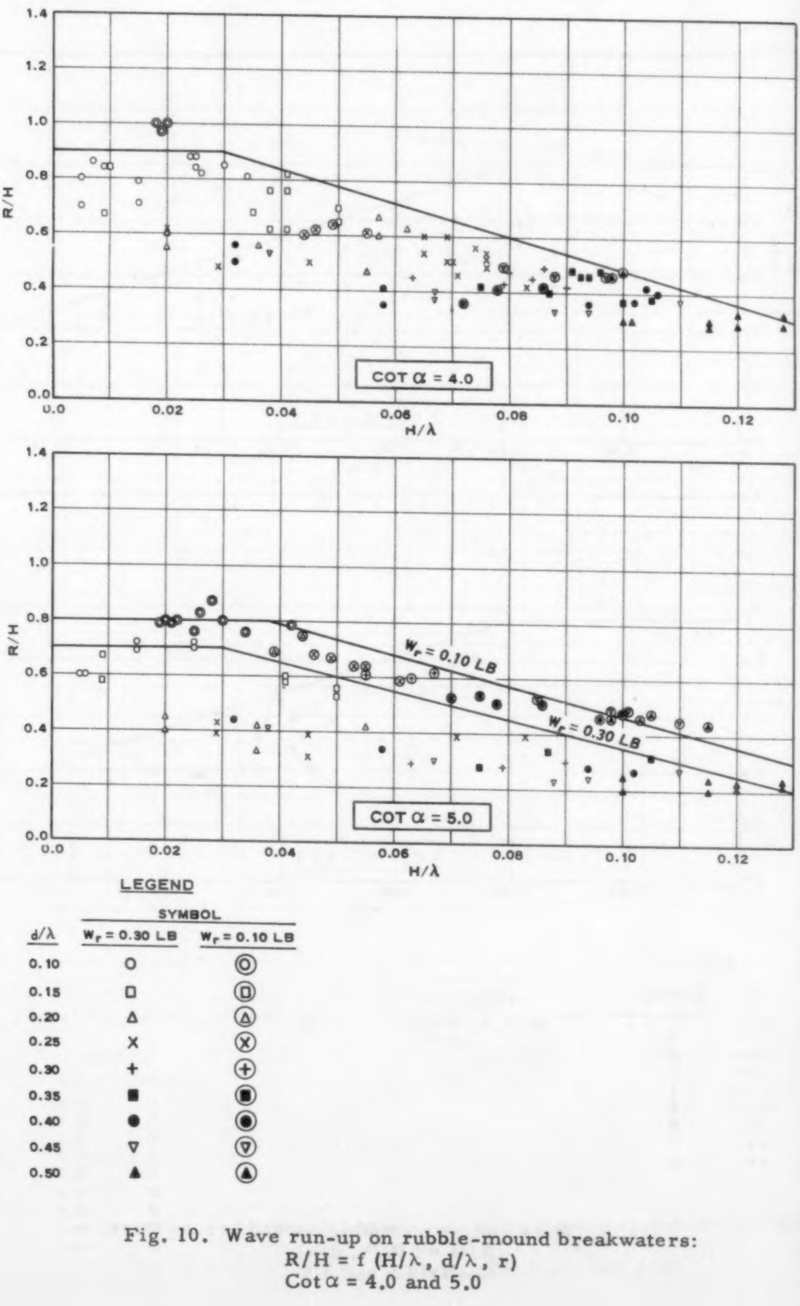

Fig. 10. Wave run-up on rubble-mound breakwaters:
R/H = f (H/λ, d/λ, r)
Cot α = 4.0 and 5.0

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type tested. Within the range of test conditions used to date, R/H decreases when either cot α or H/λ is increased.

The tests were not designed to study the effects of the hydraulic roughness of the breakwater surface on wave run-up. However, two sizes of armor stones were used in tests of sections with side slopes of 1 on 4 and 1 on 5. Results show that size of stones does not affect wave run-up on a slope of 1 on 4. For the 1-on-5 slope, however, the run-up factors for the smoother surface, i.e., the slope composed of 0.10-lb armor stones, averaged approximately 20 per cent greater than the corresponding run-up factors for the slope composed of 0.30-lb stones. This can probably be explained by the fact that waves tend to break more readily on flatter slopes, and as the breaking waves rush up the slope, the depth of flow decreases, resulting in a greater percentage of energy loss for the rougher surface. Also, the flatter slopes provide a greater distance over which the losses of energy may occur. However, these tests are not sufficient to determine fully and accurately the effects of hydraulic roughness on wave run-up, and additional tests are therefore necessary.

A qualitative measure of the effects of porosity can be obtained by comparing the results of the present tests with results of those conducted at the Waterways Experiment Station during 1954 and 1955 in an investigation of wave run-up on Lake Okeechobee levee slopes.(18) The porosity of the armor-stone cover layers used in the present tests averages about 41 per cent. Levee slopes used in the Lake Okeechobee tests were smooth and impervious. The comparison of the results of these two sets of tests showed that the run-up factor for the smooth impervious slopes averages about twice that obtained for the comparatively rough, porous slopes used in the tests of rubble-mound breakwaters.

Thickness and Porosity of Cover Layers

Breakwater design requires, in addition to quantitative data to insure stability of armor units and prevent excessive overtopping, accurate information concerning the thickness and porosity of the cover layer as functions of shape, weight, and specific weight of the individual armor units. The thickness of a layered pile of quarry stones or other type armor units may be computed by the equation

$$r = n\, k_{\Delta} \left(\frac{W_r}{\gamma_r}\right)^{1/3} \tag{22}$$

where r is the thickness of n layers of armor units of weight W_r and specific weight γ_r. The experimental thickness coefficient k_{Δ} is a function of armor-unit shape and, to some extent, the manner of placing armor units. The porosity of a given number of layers of armor units of given shape Δ, weight W_r, and specific weight γ_r can be determined by the equation

$$P = \left(1 - \frac{N_r\, W_r}{A\, \gamma_r\, r}\right) 100 \tag{23}$$

where P is the porosity in per cent, and N_r is the experimentally determined number of armor units for a given surface area, A. Eqs. (22) and (23) may also be used to estimate the thickness and porosity of underlayers.

The preparation of cost estimates and the necessary planning for construction of breakwaters are facilitated if the number of armor units required for breakwater sections of different types, and for different shapes of units, is known. The required number of armor units for a full-scale breakwater can be determined from the equation

$$N_r = A\, n\, k_\Delta \left(1 - \frac{P}{100}\right)\left(\frac{\gamma_r}{W_r}\right)^{2/3} \tag{24}$$

Tests to determine k_Δ and P as functions of armor-unit-shape have been conducted using tetrapods and quarry stones of seven different shapes (designated A through G) varying from nearly round to flat. The shapes of the rocks were determined by measuring their average dimensions in three mutually perpendicular planes. The rocks were placed pell-mell, by layers, in a square box 2 ft wide and 1 ft high. The surface of each layer was sounded to determine its average thickness, and the number of rocks required to form each layer was counted. The thickness coefficient (k_Δ) and the porosity of the rock layers (P) were then calculated by means of Eqs. (22) and (23).

Thickness and porosity data were obtained for one, two, three, and four layers of each shape of rock. Individual stones of each type having approximately the same weights and shapes were selected. The rocks varied in weight from 0.12 to 0.46 lb, and had an average specific weight of 176.0 lb per cu ft. The manner of placing the rounder rocks (shapes A, B, and C) corresponded to pell-mell construction. For the more elongated rocks (shapes D to G), the manner of placement corresponded roughly to masonry-type construction, with the largest dimension of the rock parallel to the breakwater slope. This manner of placing elongated stones was used to determine the effects of shape factor on the coefficients k_Δ and P, and is not recommended for full-scale breakwater construction.

Tests of tetrapod armor units were made using the 2-ft-square box as described above, with the units placed pell-mell, two layers thick. Tests of tetrapods were also made in which the two layers were placed in the cover layer of a breakwater test section in a more dense and geometrical pattern.

The results of tests to determine the thickness coefficient (k_Δ) and the porosity (P) for the different shapes of quarry-stone armor units are shown in Table 4. Both k_Δ and P vary with shape of the stones; neither, however, vary with the number of stone layers (n). The thickness coefficient has an average value of 0.94 for the shape A (nearly round) armor stones, and decreases as the shape of the stones becomes flatter and more elongated, to an average value of 0.64 for the shape G stones. Porosity increases as the shape of stones becomes flatter and more elongated. The average values of P for the shape A and shape G stones are 39% and 47%, respectively.

The results of tests to determine values of k_Δ and P for tetrapod armor units are shown in Table 5. For tetrapods placed geometrically, two layers thick, average values of k_Δ and P are 1.06 and 43% respectively. For tetrapods placed pell-mell, two layers thick, the respective values are 1.00 and 52%. In addition, results of Danel[7] and Hudson and Jackson[9] are shown for tetrapods placed pell-mell and placed semipell-mell, respectively. It is believed that values of k_Δ and P of 1.0 and 50% are representative of the conditions which would obtain when placing tetrapods in two layers to form cover layers of full-scale breakwaters.

Table 4

Shape and Porosity Characteristics of Quarry-stone Armor Units

Number of Layers, n	Characteristics of Stone			
	x/z	y/z	k_Δ	P, %
		Stone Shape A		
1	1.5	1.2	0.95	38
2	1.5	1.2	0.95	40
3	1.5	1.2	0.93	41
4	1.5	1.2	0.91	38
		Average	0.94	39
		Stone Shape B		
1	1.6	1.3	0.95	44
2	1.6	1.3	0.93	41
3	1.6	1.3	0.92	40
4	1.6	1.3	0.93	40
		Average	0.93	41
		Stone Shape C		
1	1.6	1.3	0.92	40
2	1.6	1.3	0.91	39
3	1.6	1.3	0.92	42
4	1.6	1.3	0.91	41
		Average	0.92	40
		Stone Shape D		
1	1.7	1.4	0.89	43
2	1.7	1.4	0.92	45
3	1.7	1.4	0.91	43
4	1.7	1.4	0.89	42
		Average	0.90	43
		Stone Shape E		
1	2.2	1.5	0.81	43
2	2.2	1.5	0.81	42
3	2.2	1.5	0.81	42
4	2.2	1.5	0.80	41
		Average	0.81	42
		Stone Shape F		
1	2.6	1.5	0.76	46
2	2.6	1.5	0.77	47
3	2.6	1.5	0.75	45
4	2.6	1.5	0.75	45
		Average	0.76	46
		Stone Shape G		
1	3.3	2.5	0.62	49
2	3.3	2.5	0.65	45
3	3.3	2.5	0.66	48
4	3.3	2.5	0.64	46
		Average	0.64	47

Table 5

Shape and Porosity Characteristics for Tetrapod Armor Units

n	k_Δ	P, %	Placement	Reference
2	1.02	49	Pell-mell	Danel
2	1.06	43	Geometrical	WES data - 30 tests
2	1.00	52	Pell-mell	WES data - 5 tests
2	1.13	46	Pell-mell below swl, geometrical above swl	Hudson and Jackson
3	1.02	46		
4	0.96	46		

CONCLUSIONS

It is concluded from the results of tests completed to date on small-scale rubble-mound breakwaters with quarry-stone and tetrapod-shaped armor units that:

a. **Iribarren's formula is not sufficiently accurate to be used in designing rubble-mound breakwaters unless it is used on conjunction with values of the experimentally determined coefficient K', as a function of breakwater slope, shape of armor unit, and the other important variables discussed in this paper.**

b. **Use of the Iribarren formula in correlating the stability-test data for rubble-mound breakwaters is not feasible, because the experimental coefficient K' varies appreciably with the coefficient of friction μ, and accurate values of the friction coefficient for the different types of armor units are very difficult to obtain.**

c. **The assumptions upon which the analysis of the phenomenon of waves attacking a rubble-mound breakwater was based are sufficiently accurate for purposes of this investigation.**

d. **Results of the stability tests conducted to date for the no-damage and no-overtopping criteria are represented with sufficient accuracy by the formula**

$$W_r = \frac{\gamma_r \left(H_{D=0}\right)^3}{K_\Delta \, (S_r - 1)^3 \cot \alpha} \tag{18}$$

e. **The amount of damage that will be done to a quarry-stone cover layer of the type tested by waves larger than the selected design wave can be estimated from the results of damage tests presented in this paper.**

f. **The safety factor for rubble-mound breakwaters with quarry-stone armor units and n > 2, designed in accordance with Eq. (18) using K_Δ = 3.2, is adequate; however, in view of the fact that nature wave trains contain waves of heights as large as 1.6 $H_{1/3}$ approximately one**

per cent of the time, compared with a corresponding value of 1.1 $H_{1/3}$ for the small-scale test waves, there is some doubt as to which of the various wave heights in natural wave trains should be selected as the design wave.

g. Eq. (18), with a value of 8.3 for K_Δ, can be used to design tetrapod cover layers for rubble-mound breakwaters; but, since preliminary tests have indicated that tetrapod cover layers with n = 2 are damaged appreciably by waves slightly larger than 1.3 $H_{D=0}$, it is recommended that design-wave heights for breakwaters having this type of cover layer be selected with caution.

h. For the conditions tested, in which the H/d ratio was comparatively small, the stability of rubble-mound breakwaters is not appreciably affected by variations in the d/λ and H/λ ratios. However, special stability tests recently completed concerning a breakwater at Nawiliwili Harbor, Kauai, T. H.,[19] where the H/d ratio is critical and waves break directly on the breakwater slope, showed that the ratios H/λ and d/λ are important variables for these conditions.

i. Two layers of armor units are optimum for tetrapod cover layers.

j. Breakwater slope (tan α) and wave steepness (H/λ) are the primary variables affecting wave run-up on rubble-mound breakwaters where the H/d ratio is sufficiently large so that breaking waves do not occur on or seaward of the breakwater slope; wave run-up decreases when values of either H/λ and cot α are increased.

k. The thickness of cover layers and the number of armor units required to cover exposed slopes of rubble-mound breakwaters can be determined by the equations

$$r = n\, k_\Delta \left(\frac{W_r}{\gamma_r} \right)^{1/3} \tag{22}$$

and

$$N_r = A\, n\, k_\Delta \left(1 - \frac{P}{100} \right) \left(\frac{\gamma_r}{W_r} \right)^{2/3} \tag{24}$$

Conservative values of k_Δ and P for selected quarry-stone armor units placed pell-mell are 1.0 and 40%, respectively. Corresponding values of k_Δ and P for tetrapods are 1.0 and 50%, respectively.

Tests in Progress

Tests are currently being conducted at the Waterways Experiment Station to determine the relative efficiencies of quarry-stone, tetrapod, tribar, tetrahedron, and other special-shape armor units. Tribars[20,21] were developed by R. Q. Palmer of the U. S. Army Engineer District, Honolulu, T. H.

As of this date (April 1959), test results obtained at the Waterways Experiment Station indicate that, with n = 2: (a) tetrahedrons are inferior to both tetrapods and tribars with respect to stability; (b) tribars are slightly better than tetrapods with respect to stability; (c) a tribar cover layer has a slightly higher porosity than a tetrapod cover layer; and (d) a smaller number of

tribars are required for a two-layer cover for rubble-mound breakwaters. Also, it has been determined that tribars can be placed as a one-layer unit above still-water level in such a way that the stability provided is considerably greater than the stability provided by two layers of either tetrapods or tribars.

ACKNOWLEDGMENTS

The experiments reported herein were performed in the Wave Action Section, Hydrodynamics Branch, of the Hydraulics Division, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, by Mr. R. A. Jackson, Hydraulic Engineer, with the aid of various assistants. Their work was under the immediate supervision of the writer. The testing program is sponsored by the Office, Chief of Engineers, U. S. Army, Washington, D. C., under Civil Works Investigation 815, "Stability of Rubble-Mound Breakwaters." The aid of Mr. J. G. Housley, Hydraulic Engineer, Waterways Experiment Station, in the preparation of material and review of this paper is greatly appreciated.

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APPENDIX

List of Symbols

a	Coefficient in Eqs. (17), (18), and (19)
Subscript a	Refers to area
A	Surface area, ft^2
b	Exponent in Eq. (17)
Subscript b	Refers to breaking-wave conditions
C	Coefficient
d	Water depth, ft
Subscript d	Refers to drag
D	Damage parameter, per cent
Subscript D	Refers to damage of cover layer
f	Reads "function of"
F	Force, lb
g	Acceleration due to gravity, ft/sec^2
h	Height of breakwater crown above swl, ft
H	Wave height, i.e. the vertical distance from trough to crest, measured at the location of a proposed breakwater, ft
k	Coefficient
K'	Coefficient in modified Iribarren formula
K_Δ	Coefficient in new breakwater stability formula for conditions of no damage and no overtopping, varies with shape of armor unit
K_D	Coefficient in new breakwater stability formula, varies with percentage of damage to cover layer for a given shape armor unit
ℓ	Characteristic linear dimension of armor unit, ft
m	Width of breakwater crown, ft
Subscript m	Refers to inertia
n	Number of layers of armor units
N	Number
P	Porosity of cover layer, per cent
Subscript q	Refers to total
r	Thickness of cover layer, measured perpendicular to slope of breakwater face, ft
Subscript r	Refers to armor unit

R	Wave run-up, vertically above swl, ft
$\mathfrak{R}$	Reynolds number
Subscript s	Refers to stability
S	Specific gravity, e.g., $S_r = \gamma_r/\gamma_w$
t	Time, sec
T	Wave period, sec
Subscript v	Refers to volume
V	Velocity, ft/sec
Subscript w	Refers to water
W	Weight, lb
W'_r	Buoyant weight of armor unit, lb
x	Abscissa
y	Ordinate
z	Vertical distance, measured positively upward from swl, ft
α	Angle of breakwater slope, measured from horizontal, deg
β	Angle of obliquity of wave attack, deg, e.g., when wave crest is parallel to breakwater alignment, $\beta = 0$
γ	Specific weight, lb/ft^3
Δ	Shape of armor unit
Subscript Δ	Refers to shape factor
λ	Wave length, ft
μ	Coefficient of friction
σ	Angle of beach slope, measured from horizontal, deg
ϕ	Angle of repose of armor units, deg
∂	Partial differential symbol
$\cot \alpha$	Reciprocal of breakwater slope
$\tan \alpha$	Breakwater slope
R/H	Wave run-up factor
d/λ	Relative depth
H/λ	Wave steepness
H/d	Relative height
$\frac{\ell}{V^2}\frac{\partial V}{\partial t}$	Form of Iversen's modulus
swl	Still-water level

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DESIGN OF SEAWALLS AND BREAKWATERS

Ira A. Hunt, Jr.,[1] M. ASCE

ABSTRACT

This paper discusses seawall and breakwater design from the standpoint of the manner and effectiveness with which they reflect or dissipate the incident wave energy. Various laboratory results are considered and their implications reconciled, particularly as regard wave up-rush.

FOREWORD

This paper on the fundamentals of wave energy dissipation, and in particular the wave-up-rush, was originally written in August 1955, at the request of Mr. A. L. Cochrane, Chief, Hydrology and Hydraulic Branch, Office of the Chief of Engineers. The conclusions have been verified by laboratory investigations. The laboratory work which was personally observed was performed in the Laboratorie Dauphinois d'Hydraulique, Grenoble, France, and in the Beach Erosion Board, Corps of Engineers. The tests at the Beach Erosion Board were under the supervision of Mr. Thorndike Saville, Jr. The original paper has been revised while at the U. S. Lake Survey for use in the Survey Report on the Water Levels of the Great Lakes. This paper has been communicated to the Sixth Conference on Coastal Engineering at Miami, Florida, in December 1957 and to the Mechanical Engineering Division of the National Research Council of Canada at Ottawa, in June 1958.

The building of protective sea walls and breakwaters along coastal and inland shores has become imperative in many places because of our rapidly increasing population and expanding industries. Economics has dictated that

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1. Major, Corps of Engrs., U. S. Lake Survey, Detroit, Mich.

these structures cannot be overdesigned, yet the impelling need for protecting life and property makes it essential that they be safely designed. Destructive forces of water masses have been cause for concern to many countries for centuries. The attention of the world was focused on the Netherlands and England when the storm flood of 1 February 1953 breached their dikes and sea walls causing great loss of life and damage to property. The author had the invaluable experience of assessing firsthand the causes of the dike failures, of which a detailed report was submitted to the Chief of Engineers, U. S. Army.[1] The sight of fertile lands and deserted villages inundated as far as the eye can see makes a lasting impression of the vital need for proper forecasting techniques and design of sea defenses, see Fig. 1. Although some of the sea walls failed because of mechanical defects and improper drainage, it is reliable knowledge that the majority of all failures were caused by the overflowing or overtopping of the sea walls. The structures were stable until this overtopping occurred and, generally, it was the erosion of the land faces that brought about the failures, see Fig. 2. It is, therefore, of the utmost importance that the engineer have a working knowledge of the expected wave uprush to properly design safe hydraulic structures.

Wave Characteristics

Because waves are the prime destructive force, to design a safe structure, facts must be obtained about the expected waves and their action. In some regions, the transfer of the energy of the wind to the water surface causes a rise in the water level. In this paper, I have assumed that the storm water level is known; that is, the combination of normal water level, tide, and wind set-up.

Most waves are generated at sea in what is called deep water where the bottom has no affect on the normal wave properties. Usually, protective structures are located on the shore line or in shallow water. Thus the bottom affects the waves and alters their characteristics before they reach the structure. Deep water wave characteristics can be computed using the fetch type formula of Sverdrup and Munk.[2] The characteristics of waves generated in very shallow water can be obtained by using the relationships given in the Corps of Engineers publication "Waves and Wind Tides in Shallow Lakes and Reservoirs."[3]

It is necessary to define the meaning of the symbols used in this paper. Most of these symbols are shown in Fig. 3.

There are several relationships, all very well described in the Beach Erosion Board Technical Report No. 4,[4] which will be discussed briefly to assist in clarifying the change in wave properties as a wave is propagated from deep to shallow water. The term "deep water" usually connotes water deeper than one half the surface wave length, $\frac{D}{L} > \frac{1}{2}$.

Fig. 4 shows the relationship between shallow water and deep water wave heights as a function of $\frac{D}{L_o}$. It can be seen that for values of $\frac{D}{L_o} > .033$ there is never more than a 10 per cent variation from unity in the value of $\frac{H}{H_o}$.

The wave length can be computed as

$$\frac{L}{L_o} = \tanh \frac{2\pi D}{L} \tag{1}$$

Figure 1

Town of Kerkwerve on the island of Schouwen, the Netherlands. Waves attacking buildings in inundated polder.

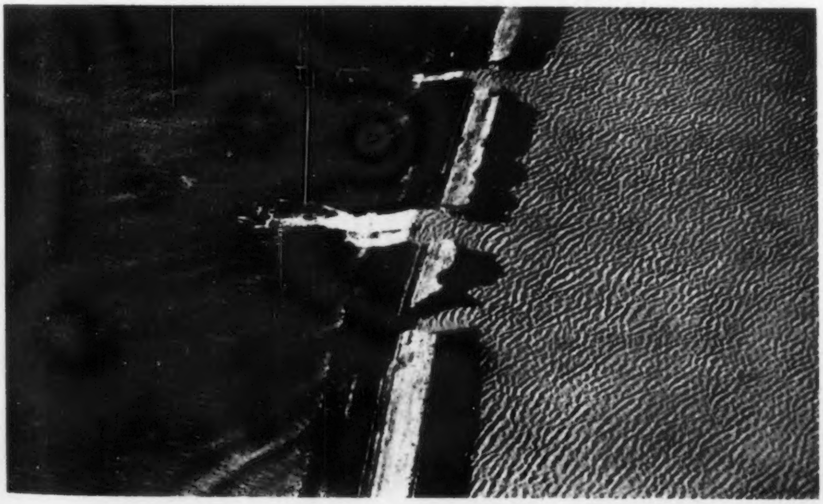


Figure 2

Example of the failure of the inside face of a dike. Notice the different wave action in the shallow water of the polder.

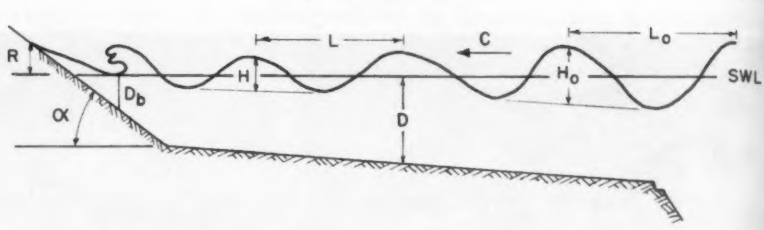


Figure 3. Definition of symbols

R	Maximum vertical distance above storm water level of the wave up-rush.
α	The angle of inclination of the structure with the horizontal.
H	Wave height at a given point.
L	Wave length at a given point.
T	Wave period.
C	Wave velocity.
D	Water depth from bottom to the storm water level.
SWL	Storm water level.
	The subscripts o refer to deep water wave conditions.
	The subscripts b refer to the wave condition at the point of breaking.

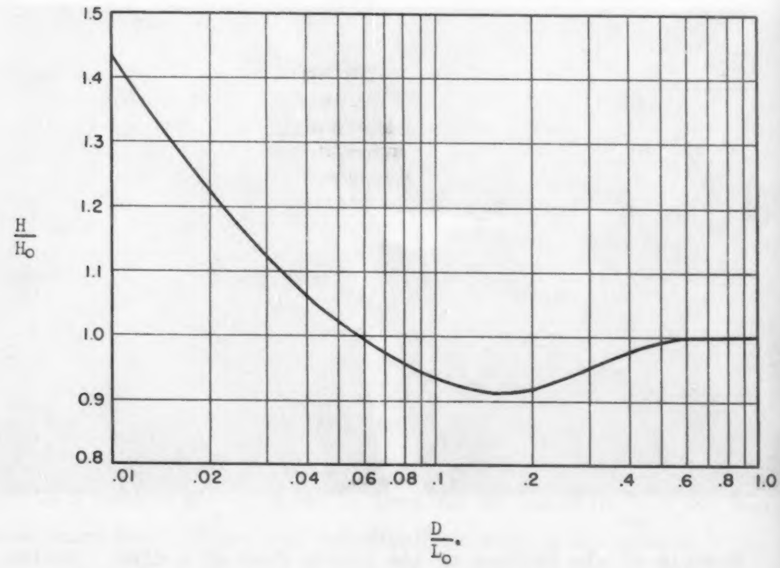


Figure 4. Relation of shallow water wave height, H, to deep water wave height H_o, for varying values of $\frac{D}{L_o}$

Munk[5] has shown in his paper on solitary waves that

$$\frac{H_b}{D_b} = .78 \tag{2}$$

Suquet[6] has calculated the relationship between $\frac{D_b}{H_o}$ and $\frac{H_o}{L_o}$. This is shown in Fig. 5.

With these relationships and the general formula for the velocity of a wave

$$C^2 = \frac{gL}{2\pi} \tanh \frac{2\pi D}{L} \tag{3}$$

it is possible to compute most of the necessary wave characteristics. It should be remembered that the wave period, T, may be considered constant as the wave approaches the shore.

In this paper, it will be presumed that the waves approach the structure frontally and the wave heights are unaffected by refraction. However, if the structure or coast line is fronted by a gentle sloping beach, obliquely approaching waves will be bent so that the wave crests will align themselves with the bottom contours and the waves will advance almost frontally.

The velocity at which the energy of a wave is propagated shoreward differs from the wave velocity. In deep water, the wave energy moves with one half the wave speed, whereas in very shallow water—that is, when $\frac{D}{L} < .04$—the energy is propagated with the speed of the waves. The total energy of a shallow water wave is slightly less than that of a deep water wave. The energy

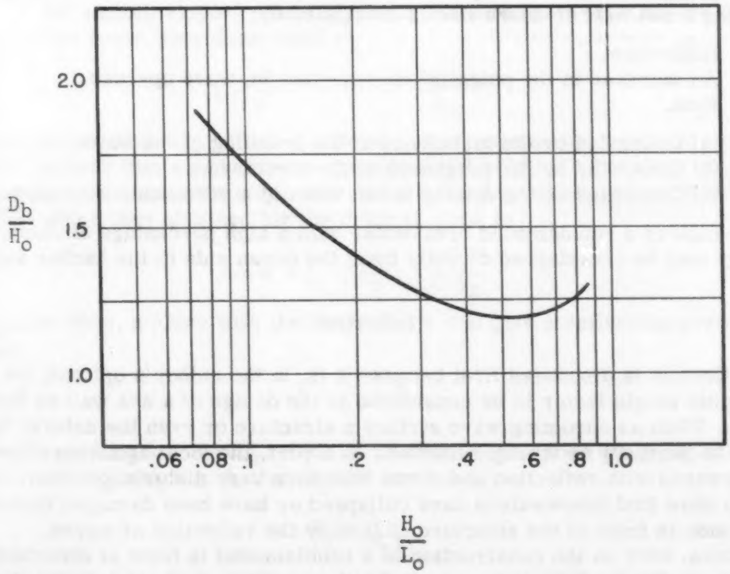


Figure 5. Relationship of the ratio of the depth of breaking to the deep water wave height, $\frac{D_b}{H_o}$, with the deep water wave steepness $\frac{H_o}{L_o}$, from Suquet[6]

of a shallow water wave is concentrated closer to the water surface than it is for a deep water wave. Gaillard[7] has shown that for a wave $\frac{H}{L} = \frac{1}{15}$ and $\frac{D}{L} = \frac{1}{10}$ the total theoretical energy of the shallow water wave is 95 per cent of that of a deepwater wave of equal height and length; yet in the case of the shallow water wave, an amount of energy equal to more than half the total energy of the deep water lies above the horizontal plane situated at a distance of .03L below the line of surface orbit centers, while in the case of the deep water wave only about 31 per cent of the total energy of the wave lies above the same plane.

Thus we can see that the wave characteristics and the depth of the water at the approach of a structure have a great part to play in the forward transfer of energy and the concentration of energy in the wave. In 1953 the author summarized all the general formulae known to him which were being utilized to compute wave up-rush.[1] One fact was outstanding—wave characteristics, except for the approaching wave heights, did not enter into these formulae. This paper will emphasize the importance of wave characteristics and demonstrate how they affect the design of sea walls and breakwaters.

Energy Dissipation

The key to proper design is an insight into the manner in which the oncoming wave energy is dissipated. In general terms, the energy of a wave striking a sea wall or shore line is dissipated by:

1. Reflection.
2. An increase in the potential energy—that is, wave up-rush.
3. Heat.
   (a) Generated by the turbulence of the breaking of the wave.
   (b) Generated by the roughness of the structure.
   (c) Generated by the mixing in the voids of a permeable structure.

In the case of a rubblemound breakwater with a high percentage of voids, the energy may be transferred directly from the ocean side to the harbor side.

Reflection

Reflection is discussed first because it is, in the author's opinion, the most important single factor to be considered in the design of a sea wall or breakwater. When an oncoming wave strikes a structure or even the natural coast, it can be partially or wholly reflected. In a port, the local agitation of the sea is increased with reflection and it can become a very disturbing factor. Accounts show that breakwaters have collapsed or have been damaged because of erosion in front of the structure caused by the reflection of waves. Measures, such as the construction of a rubblemound in front of structures, have been effected to reduce wave reflection and thereby increase the stability of the bottom. If it is at all possible, the reflection should be kept to a minimum.

Excellent papers discussing reflection have been written by M. Miche[8,9] and Iribarren and Nogales.[10] Miche has determined the maximum value of the wave steepness at sea, $\left(\frac{H_o}{L_o}\right)_{max}$ or γ_o max, of a wave which is

theoretically capable of being totally reflected by an indefinite inclined plane making the angle α with the horizontal as

$$\gamma_{o\ max} = \sqrt{\frac{2\alpha}{\pi}} \quad \frac{\sin^2\alpha}{\pi} \tag{4}$$

If the profile of the wave is considered, the maximum steepness of the wave $\frac{dy}{dx}$ is equal to tan α. For a greater steepness there will be breaking of the incident wave or the dissipation of power preventing total reflection. He defines the reflecting power, R, of a structure or of a coast as the ratio of the amplitude H_1 of the reflected swell to the amplitude H of the incident swell

$$R = \frac{H_1}{H} \tag{5}$$

If γ_0 is the wave steepness at sea and $\gamma_{0\ max}$ is the steepness compatible with total reflection, the part theoretically reflected, R', has the value

$$R' = \frac{\gamma_{o\ max}}{\gamma_o} \qquad R' < 1 \tag{6}$$

According to results, Miche states that it should be expected that there is a certain diminution in volume of the reflected swell due to the turbulence produced in the orbital movement of the wave in contact with the slope. Thus, there is reason to introduce a coefficient ρ for intrinsic reflection of the slope, apparently independent of the slope and which should have a value of 0.8 or 0.9 for smooth slopes. The effectively reflected part—that is, the reflecting power R—is, therefore equal to

$$R = \frac{H_1}{H} = \rho R' = \rho \frac{\gamma_{o\ max}}{\gamma_o} \tag{7}$$

Iribarren and Nogales, utilizing different assumptions, have defined a critical slope as such that a reduction in the slope will produce a breaking of the wave and an increase in the slope will cause a surging action and reflection. The value which they obtained for the critical slope is

$$\tan\alpha = i = \frac{8}{T} \sqrt{\frac{H}{2g}} \tag{8}$$

For a given wave, a slope with the above value will give a reflecting power of one half.

$$R = \frac{H_1}{H} = \frac{1}{2} \tag{9}$$

Should one assume that $H \approx H_0$, which is a logical assumption for most actual shore conditions, see Fig. 4, then Eq. (8) becomes

$$\gamma_o = \frac{i^2}{5.1} \tag{10}$$

Miche states that his formula is less accurate for very slight slopes than for the steeper slopes. If we consider values of $\alpha \geq 11^o$—that is, $\tan\alpha = i \geq \frac{1}{5}$—the value of $\gamma_{0\ max}$ can be approximated from Eq. (4) by

$$\gamma_{o\ max} = \frac{i^2}{8.1} \tag{11}$$

Now $\frac{H_1}{H} = \rho \cdot \frac{\gamma_{o\,max}}{\gamma_o}$, but the γ_o of Iribarren and Nogales is for the condition $\frac{H_1}{H} = \frac{1}{2}$.

Therefore, substituting the values of Eqs. (10) and (11),

$$\frac{H_1}{H} = \frac{1}{2} = \rho \quad \frac{\frac{i^2}{8.1}}{\frac{i^2}{5.1}} \tag{12}$$

$$\rho = \frac{8.1}{2(5.1)} = .8$$

And, since $\rho = .8$ in Eq. (12), the results of Iribarren and Miche are comparable. For an impermeable smooth slope, Miche has stated that the value of ρ should be approximately 0.8. The formula of Iribarren was derived for a smooth impermeable slope.

It was previously stated that sea walls and breakwaters should be constructed to keep the reflection of incident waves to a minimum. It has been shown theoretically that if $\gamma_o > \frac{i^2}{5.1}$, breaking will occur and the reflecting power will be less than one half. Therefore, it is of importance to determine if this criteria of Iribarren and Nogales holds true in actual cases. They conducted a few experiments which are summarized in the following table.

Table 1

Laboratory Data of Iribarren and Nogales*

Wave Characteristics		Measurement of Slopes $i = \tan \alpha$			Computed Slopes (Iribarren)	Computed Slopes (Miche)
H_{cm}	T_{sec}	Total Breaking	Total Reflection	Average	$i = \frac{8}{T}\sqrt{\frac{H}{2g}}$	$\gamma_{o\,max} = \sqrt{\frac{2\alpha}{\pi}} \cdot \frac{\sin^2\alpha}{\pi}$
5.5	0.66	0.42	0.86	0.64	0.66	0.80
4.5	0.92	0.29	0.59	0.44	0.42	0.50
4.5	1.00	0.33	0.49	0.41	0.38	0.46

* M. Miche, Reference 9, Table 1

It can be noted that the formula of Iribarren and Nogales gives results which are very close to the average value between total breaking and total reflection, and that the formula of Miche gives excellent results for the slope necessary for total reflection.

Caldwell, in his article "The Design of Wave Channels",(11) states that

> It was noticed that as the test slope is steepened, at some point a troublesome amount of energy would be reflected from the slope in the form of a reflected wave. An investigation of this action showed that for a 6-foot, 7-second wave (prototype) a 1:3 slope destroyed sufficient energy so that no reflection problem was in evidence. It was further found that for a 6-foot, 14-second wave a slope of 1:5 could be tested without excessive reflection.

If the formula of Iribarren, for given wave characteristics, will give the slope which will not give excessive reflections, then the answers given by his formula should be comparable to those quoted above. Table 2 shows that the answers are comparable.

Table 2

Laboratory Data of Caldwell[11]

Wave Characteristics		Slopes Determined in the Laboratory	Slopes Calculated by $i = \sqrt{\frac{H}{T^2}}$
H_{ft}	T_{sec}		
6	7	.333	.349
6	14	.200	.175

Granthem(12) performed some of the first tests on wave up-rush in the United States. In his laboratory tests, he noted whether the wave broke on the structure or whether the wave surged up the structure surface without breaking. Fig. 3 of his paper depicts four graphs of $\frac{R}{H}$ versus the slope of structure for constant values of $\frac{D}{L}$ and $\frac{H}{L}$. The value of slope given by $i = \sqrt{5.12 \gamma_0}$ definitely appears to be the limiting slope between the breaking and surging of waves on the structure. The data in Table 3 was derived from Granthem's paper. This table indicates that Iribarren's formula computes the limiting slope remarkably well.

Table 3

Laboratory Data of Granthem[12]

Wave Characteristics		Longest Measured Slope Where Breaking Occurred (degrees)	Slope Computed by Iribarren's Formula (degrees)	Smallest Measured Slope Where Surging Occurred (degrees)
D/L	H/L			
0.066	0.012	--	21.5	15
0.148	0.035	25	26.5	30
0.218	0.071	30	33.0	36
0.434	0.112	35	37.0	45

The Beach Erosion Board is at present conducting a systematic investigation of wave up-rush. The author applied the criterion of Iribarren to dozens of tests that they made in their wave channels and found conclusive evidence that $i = \sqrt{\frac{H}{T^2}}$ is approximately the critical slope for a smooth impermeable structure—that is, for a given wave, a decrease in the slope will definitely cause the wave to break on the structure.

It may be concluded, then, that there is excellent evidence to indicate that the value of the slope given by the formula of Iribarren and Nogales

$$i = \frac{8}{T}\sqrt{\frac{H}{2g}} \approx \sqrt{\frac{H}{T^2}} \tag{13}$$

is indeed the limiting slope between breaking and surging waves. In design, therefore, it is most desirable to have the slope of the structure, i, such that it is less than $\sqrt{\frac{H}{T^2}}$ for the design waves. In this way, one is assured of reduced reflections and increased dissipation of energy by the heat generated by the turbulence of the breaking wave.

Wave Up-Rush

Another method by which oncoming wave energy is dissipated is by wave up-rush. When a wave breaks and rushes up a beach or structure, some energy is dissipated by the turbulence of the breaking of the wave and the remaining kinetic energy of the wave is transformed into potential energy as it runs up the slope. In this section, only an impermeable smooth structure with a continuous slope will be considered. Once the wave up-rush on this simplified type of structure has been analyzed, an attempt will be made to predict what will happen with rough, porous, and discontinuous sloping structures.

In formulating the physical law which governs a given natural phenomena, it is normal engineering procedure to form dimensionless parameters from the variables which influence the action studied. The dimensionless parameters evolved can then be studied in the laboratory and a solution effected.

In an investigation to determine the amount of up-rush on a smooth impermeable continuous slope, the following variables should be considered:

- R - wave up-rush
- H - wave height; it is assumed $H \approx H_o$
- L - wave length
- D - depth of water
- C - wave velocity; equals $\frac{L}{T}$
- i - slope of the structure; equals $\tan \alpha$
- E - wave energy
- ρ - density of the fluid
- μ - viscosity of the fluid

Utilizing the Buckingham Pi theorem and the aforementioned variables, the following dimensionless parameters are obtained.

$$\frac{R}{H}, \frac{H}{L}, i, \frac{H}{D}, \frac{H^2 C^2 \rho}{E}, \frac{HC\rho}{\mu}$$

The term $\frac{HC\rho}{\mu}$, which is in the form of the Reynolds Number, may be neglected because the viscous effects should be small except for very flat beach slopes, which present no engineering problem.

The term $\frac{H^2 C^2 \rho}{E}$ reduces to approximately the value of $\tanh \frac{2 \pi D}{L}$; consequently,

$$\frac{R}{H} = f \left(\frac{H}{L}, i, \frac{H}{D}, \tanh \frac{2 \pi D}{L}\right) \tag{14}$$

At this point, study in the hydraulics laboratory of the effect which these parameters have upon the wave up-rush is required. This can be done easily by varying one parameter at a time while the others remain constant.

Breaking Waves.—The mechanics of breaking waves and surging waves are entirely different. It has previously been determined that whenever the slope of a structure is less than $\left(\frac{H}{T^2}\right)^{1/2}$, waves will break upon the structure. In any laboratory investigation, it is of paramount importance to differentiate between waves which break upon a structure or beach and waves which surge up the structure or beach. It is also important to note whether oncoming waves break upon the structure or upon the beach in front of the structure. The up-rush of waves which break upon the beach in front of a structure must be considered and computed as breaking on a composite slope and not a continuous slope. Composite slopes will be discussed later.

From the studies of the Waterways Experiment Station,[13] it can be shown rather conclusively that the parameter $\frac{H}{D}$ has very little, if any, effect on wave run-up as long as the waves break upon the structure, see Fig. 6. This has also been evidenced by the tests conducted at the Beach Erosion Board.[19] It should definitely not be concluded that the depth, D, has no effect on the run-up because it plays a vital role insofar as it affects the wave characteristics. It is only for waves breaking upon a structure that the dimensionless parameter $\frac{H}{D}$ is not of importance.

The same series of tests by the Waterways Experiment Station and the Beach Erosion Board measured the up-rush on impermeable structures with varying slopes for given wave conditions. The results indicated that the up-rush is definitely proportional to the slope. This is also illustrated in Fig. 6, therefore,

$$\frac{R}{H} = C \tan \alpha \tag{15}$$

For given slopes, the values of C in Eq. (15) were found to be a function of $\left(\frac{H}{L}\right)^{-1/2}$ In which case,

$$\frac{R}{H} = f \left(\frac{H}{L}\right)^{-1/2} \tag{16}$$

On the other hand, for a constant slope and wave steepness, the data of the Waterways Experiment Station, the Beach Erosion Board, and Granthem indicate that, roughly,

$$\frac{R}{H} = f \left(\tanh \frac{2 \pi D}{L}\right)^{-1/2} \tag{17}$$

From the aforementioned relationships—Eqs. (14), (15), (16), and (17)—the ratio of the up-rush to the oncoming wave height can be written as a function of the previously determined dimensionless parameters as follows:

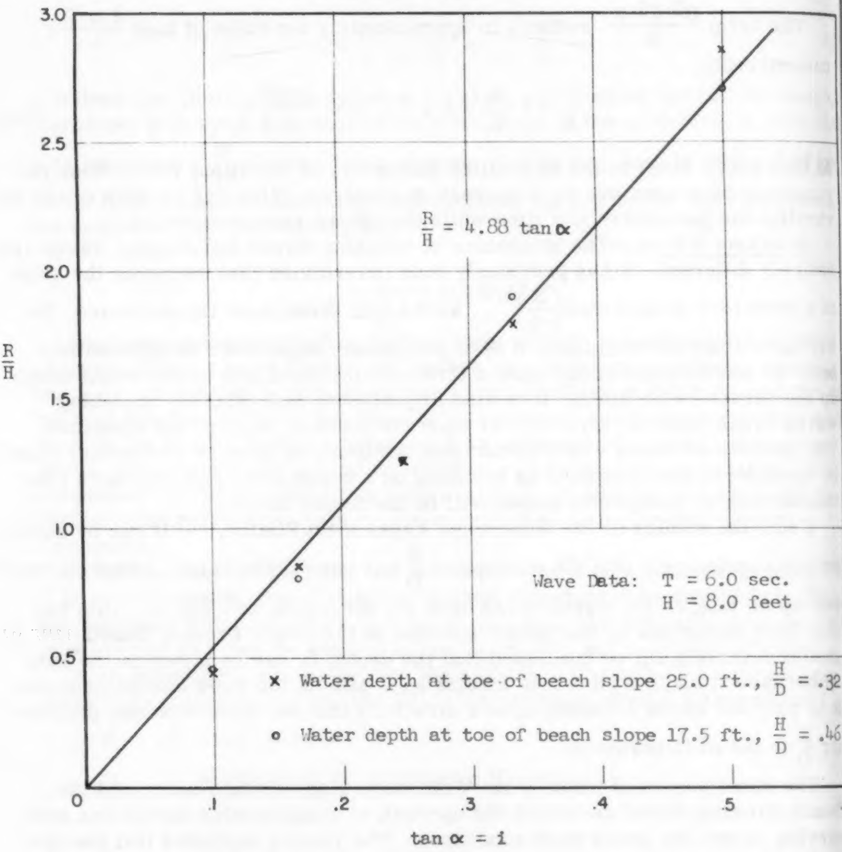


Figure 6. Ratio of wave up-rush to wave height for varying beach slopes and water depths, data from Waterways Experiment Station

$$\frac{R}{H} = K_1 \left(\frac{H}{L}\right)^{-1/2} (\tan\alpha)^1 \left(\tanh \frac{2\pi D}{L}\right)^{-1/2} \left(\frac{H}{D}\right)^c \tag{18}$$

but, since $\frac{H}{T^2} = 5.12 \frac{H}{L} \tanh \frac{2\pi D}{L}$ (19)

$$\frac{R}{H} = \frac{K \tan\alpha}{\left(\frac{H}{T^2}\right)^{1/2}} \tag{20}$$

A thorough analysis of all the experimental data available indicates that the value of K is approximately 2.3, therefore, Eq. (20) becomes

$$\frac{R}{H} = \frac{2.3 \tan\alpha}{\left(\frac{H}{T^2}\right)^{1/2}} \qquad \left\{ \begin{array}{l} \left(\frac{H}{T^2}\right)^{1/2} > \tan\alpha \\ H \approx H_o \end{array} \right. \tag{21}$$

Surging Waves.—When $\left(\frac{H}{T^2}\right)^{1/2}$ is less than tan α, the oncoming waves generally will not break on the structure but will surge up the structure. In this case, it has been shown that the reflecting power is greater than one half. Large quantities of energy are no longer dissipated by breaking, but additional energy is dissipated by reflection. The amount of energy converted into run-up is not immediately decreased, and tests show that the run-up generally increases from 2.3 times the wave height given by Eq. (21) when $\left(\frac{H}{T^2}\right)^{1/2}$ = tan α to approximately three times the wave height at sea.

$$\frac{R}{H} \approx 3.0 \tag{22}$$

It may be assumed theoretically that, when the approaching wave steepness is equal to $\gamma_{o\ max}$, 100 per cent reflection will occur. In this case, since there is no breaking of the wave, the up-rush on a vertical structure can be determined by the formula of Sainflou[14]

$$\frac{R}{H} = 1 + \frac{H}{L} \coth \frac{2\pi D}{L} \tag{23}$$

It is interesting to note that, according to the theory of Miche,[8] an inclination of 45 degrees is capable of reflecting 100 per cent of most incident waves since γ_o is rarely greater than 0.11. Miche has shown that the theoretical wave up-rush on a sloping structure can be determined by the formula

$$\frac{R}{H} = \sqrt{\frac{\pi}{2\alpha}} \tag{24}$$

The data of Granthem[12] and Sibul[15] show this formula to be approximately true. For surging waves, there generally is a decrease in the up-rush with decreasing values of γ_o for structures with slopes steeper than one half. This can be seen in Fig. 7, wherein experimental data of Granthem is depicted.

It is well known that, as the slopes decrease, the amount of energy dissipated by friction becomes increasingly great. For a given slope, there is a critical range of γ_o's where the value of $\frac{R}{H}$ is a maximum; after which the ratio will decrease with decreasing values of γ_o. In the Beach Erosion Board tests, the maximum value of $\frac{R}{H}$ was approximately five, and this was for wave steepnesses that had slight practical applications except in considering tidal waves. For slopes of tan α less than one half, the ratio $\frac{R}{H}$ will increase slightly for values of $\left(\frac{H}{T^2}\right)^{1/2}$ a little less than tan α; the ratio will then level off at its maximum value; and, finally $\frac{R}{H}$ will decrease with decreasing values of γ_o. This is clearly seen in Fig. 8.

The characteristics of storm waves are generally such that $\gamma_o > .05$; so that for steep slopes of α equal to or greater than 45°, the ratio of $\frac{R}{H}$ may be conservatively considered to be approximately 3 in design. Whereas, in design for slopes of α less than 45° it expedient to utilize Eq. (21) to compute the ratio $\frac{R}{H}$.

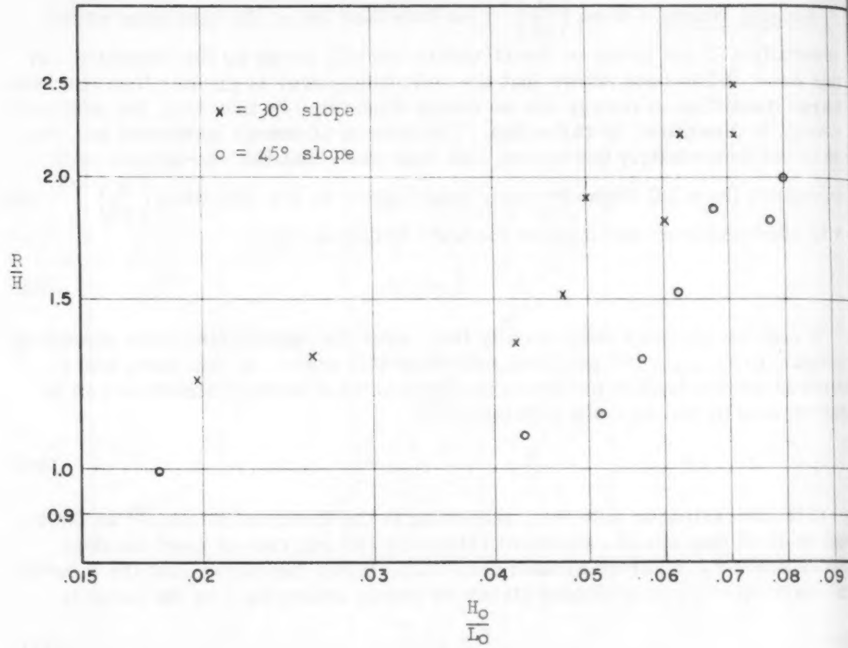


Figure 7. Ratio of wave up-rush to wave heights for surging waves and varying values of $\frac{H_o}{L_o}$, data from Granthem

Composite Slopes and Berms.—It is seldom possible to design protective works with continuous slopes. More often than not, it is advisable—for engineering and economic reasons—to utilize composite slopes or berms, see Fig. 9. New construction in England has made much use of the berm—excelle examples are the Pitt Level and Dymchurch sea walls. However, the Dutch were the first to make use of a berm on the seaward side of a sea wall and they have successfully continued to do so for years. In The Netherlands after the 1953 storm flood, two sections of a dike were observed, one with a wide berm and the other with a narrower berm, which had been exposed to the sam wave attack. The section with the wide berm stood intact, whereas the sectio with the narrower berm was destroyed even though it had a higher crown. There is no doubt that from a hydraulic standpoint a berm has advantages and it is the purpose of this portion of the discussion to analyze up-rush on smoot impermeable structures with berms.

Composite Slopes—A composite structure is a simplified case of a berm where the wave up-rush is dissipated on the second slope and, as a consequence, a third slope is not necessary. Since a breakwater or a sea wall is required to resist wave attack under all conditions, it is desirable to design for storm conditions. The ultimate design height is dependent upon the degree of protection necessary and the monies available. The slope i_1 should be so designed that all oncoming waves will break on it; that is, $\frac{H}{T^2} > i_1^2$. To insure

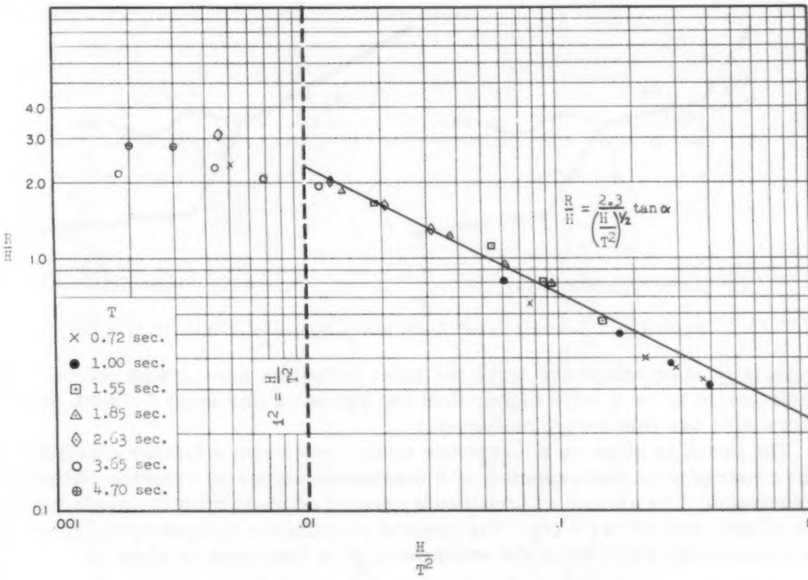


Figure 8. The ratio of wave up-rush to wave heights for varying values of $\frac{H}{T^2}$ for a continuous smooth impermeable beach slope of 1:10, data from Beach Erosion Board

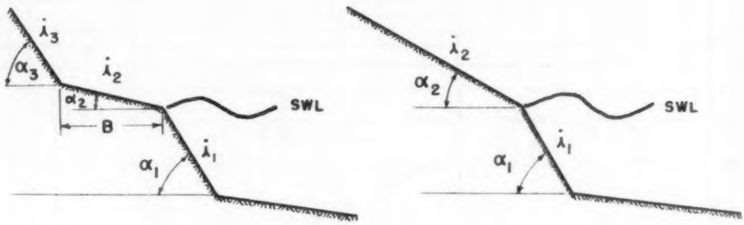
the waves striking the slope i_1, the break in slope of a composite slope structure should be at the highest expected water level. If the slope i_2 is less than i_1, a reduction in wave up-rush will occur. The curve $\frac{H}{T^2}$ versus $\frac{R}{H}$ for the composite slope will fall in between and be parallel to the run-up curves:

$$\frac{R}{H} = \frac{2.3\ i_1}{\left(\frac{H}{T^2}\right)^{1/2}} \tag{25}$$

$$\frac{R}{H} = \frac{2.3\ i_2}{\left(\frac{H}{T^2}\right)^{1/2}} \tag{26}$$

Analysis of the tests of the Waterways Experiment Station and the Beach Erosion Board clearly indicates that the foregoing statement is true, see Fig. 10.

Whenever the water level is well below the break in slope, the wave up-rush can be computed by Eq. (25). If the storm water level should be much above the break in slope so that the waves break on the upper slope, the wave up-rush can be computed by Eq. (26). For proper design, the break in slope in composite structures and the berm height in berm-type structures should be at the highest expected storm water level. This design will give the maximum dissipation of energy with the minimum construction costs. Bruun[16] has stated, "Experiments with varying berm height proved that the berm,



a. Berm-type Structure b. Composite Slope Structure

Figure 9. Schematic of Berm and Composite Slope Structures

especially when roughened, gave the least reflection when it was placed at the same height as or a little higher than the highest water level." Dutch engineers also use this design criterion.

The break in slope on a composite structure causes a further dissipation of wave energy over that expected with continuous slopes by creating additional turbulence. The amount of turbulence created depends upon the angle between the slopes; that is, $\alpha_1 - \alpha_2$. The general formula for computing the up-rush on a composite slope when the water level is at the break in slope is

$$\frac{R}{H} = \frac{2.3}{\left(\frac{H}{T^2}\right)^{1/2}} \left(\frac{\tan \alpha_1 + \tan \alpha_2}{2}\right) S \qquad (27)$$

where S is a function of $(\alpha_1 - \alpha_2)$.

In the special case where α_1 equals α_2, Eq. (27) becomes identical with Eq. (21).

With reference to Fig. 10, curves were fitted by eye through the plotted points in accordance with Eq. (27). The obtained values of S were 0.9 for slopes 1:3 to 1:6 and 0:8 for slopes 1:3 to 1:10 and 1:10 to 1:3. It is most interesting to note that the up-rush equation for the Waterways Experiment Station data on a composite structure of 1:3 to 1:10 was exactly the same as that for the Beach Erosion Board data on a composite structure of 1:10 to 1:3. This fact is extremely important, because, if the offshore conditions are such that the wave will definitely break upon the lesser slope, a great savings in material can be effected. This is illustrated in Fig. 11, where the cross-hatched area represents the savings in material gained by using a 1:10 to 1:3 composite structure instead of a 1:3 to 1:10 structure.

Berms.—Whenever local conditions are such that it is impractical to extend the upper slope, i_2, of a composite structure so that all the oncoming wave energy is dissipated on this slope, it is necessary to resort to the use of a third slope, i_3, to retain the wave up-rush, see Fig. 9. This triple-sloped structure is called a berm-type structure and the intermediary slope, i_2, is defined as the berm. The berm slope should be relatively flat, and, as previously discussed, the break in slope between i_1 and i_2 should be at the highest expected storm water level. There has been much discussion in past years about what the berm width, B, should be and to what extent a berm reduces the wave up-rush.

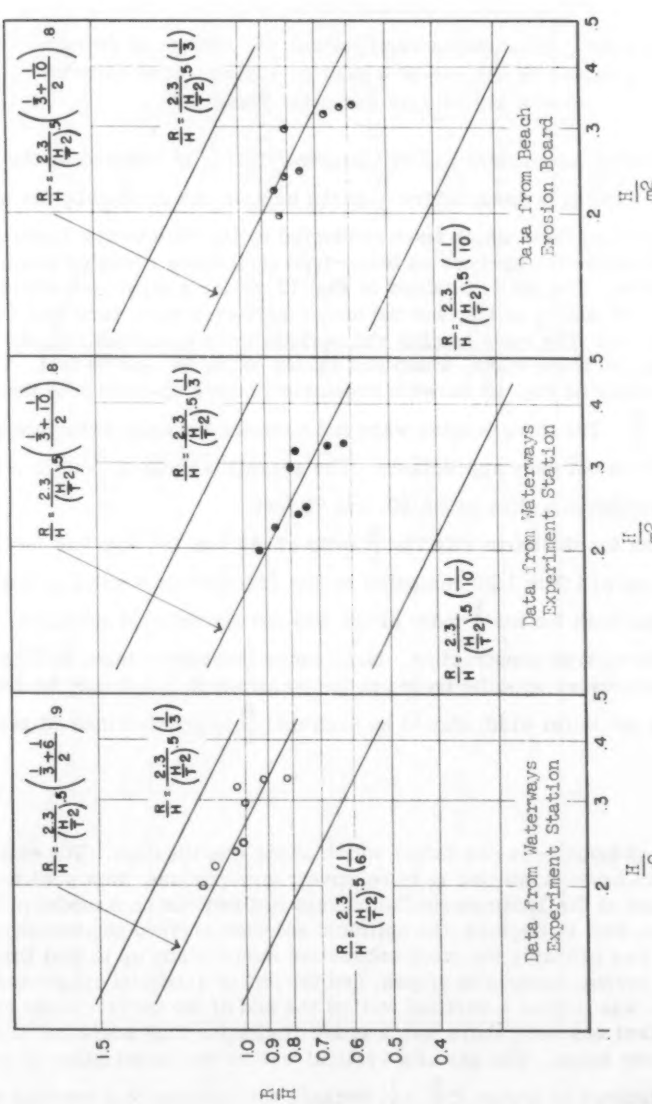


Figure 10. The ratio of wave up-rush to wave heights for varying values of $\frac{H}{T^2}$ for composite slope structures

Figure 11. Illustration representing the savings in material to be gained by the use of a 1:10 to 1:3 composite structure over a 1:3 to 1:10 composite structure

The Hydraulics Laboratory at Delft suggests that $\frac{R}{H}$ is reduced by the factor $\left(1 - \frac{B}{L}\right)$ when a berm is used, where $\frac{B}{L}$ is the berm width divided by the wave length. Analysis of tests which were conducted by the Waterways Experiment Station to determine the up-rush on berm-type structures revealed some very interesting facts. The data contained in Fig. 12, is for a structure where i_1 is 1:3, i_2 is 1:20, and i_3 is 1:3, and the oncoming waves were such that they broke on slope i_1. The wave heights and periods were kept constant, the only variable being the berm width, which had values of 30, 50, and 70 feet. The graphical analysis of Fig. 12 shows a reduction in wave up-rush with increasing values of $\frac{B}{L}$. The wave lengths were not constant for each wave type generated, but they did not vary appreciably. The average values of $\frac{B}{L}$ were .18, .31, and .43 for berm widths of 30, 50, and 70 feet.

The up-rush for the berm with the $\frac{B}{L}$ ratio of .43 was the same as for a composite slope of 1:3 to 1:20 computed by Eq. (27) with an S value of 0.8. The reduction in up-rush for the $\frac{B}{L}$ ratio of .18 was not appreciable enough to warrant the berm-type construction. Many more laboratory tests will have to be conducted before specific facts can be ascertained, but it may be said generally that the berm width should be such that $\frac{B}{L}$ is greater than 20 per cent; that is

$$\frac{B}{L} \geq \frac{1}{5} \tag{28}$$

The effect of slope i_3 is one factor which needs clarification. The experiments of Bruun have shown that i_3 is relatively unimportant. In a series of tests performed at the Laboratorie Dauphinois in Grenoble on a model of the North Kent sea wall in England, the optimum solution arrived at after performing what was probably the most exhaustive model study up to that time of the effects of berms, composite slopes, and the use of artificial roughness on wave up-rush, was to plan a vertical wall at the end of the berm. In the case of the North Kent sea wall, there was a space limitation that would not allow the use of a wide berm. The use of a vertical wall at the termination of the berm is satisfactory in design if $\frac{B}{L} > \frac{1}{5}$, because the addition of a vertical wall or a steep slope (1:3 or 1:2) at the termination of the berm creates additional turbulence with the result that additional energy is dissipated. No satisfactory quantitative solutions can be given at this time as to the effect and design of the i_3 slope. Again, further laboratory investigations are necessary.

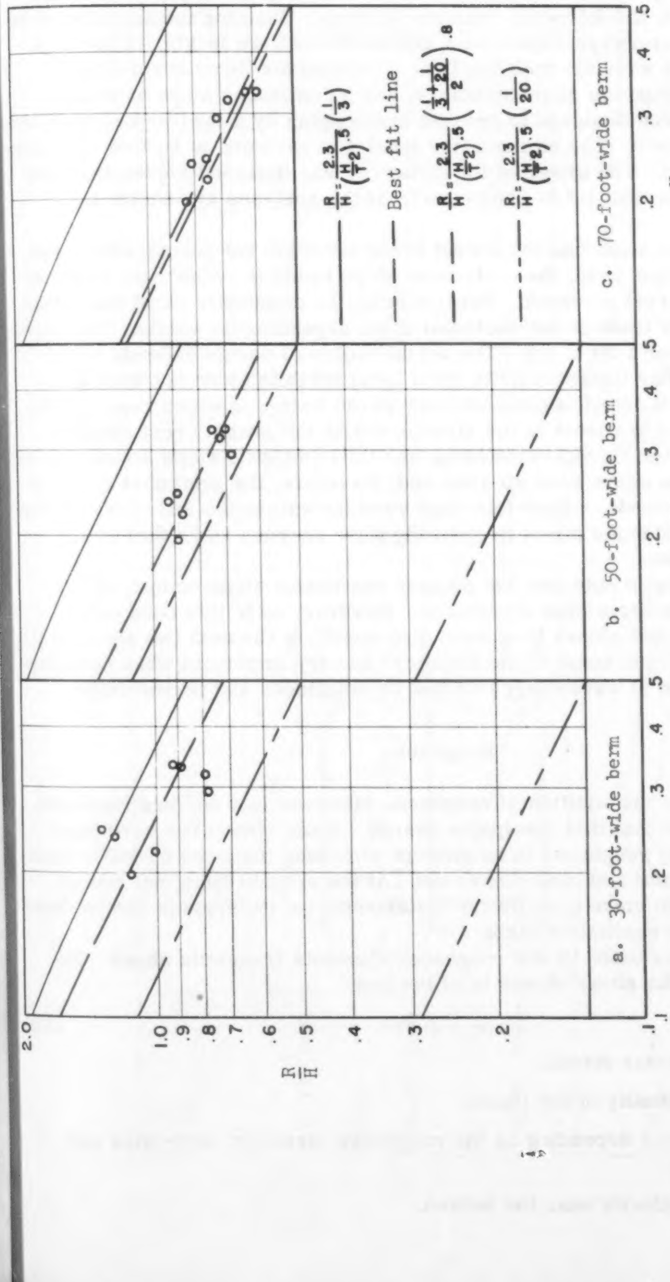


Figure 12. The ratio of wave up-rush to wave height for varying values of $\frac{H}{T^2}$ for berm-type structures of varying berm widths, data from Waterways Experiment Station

It is not difficult to visualize the great economy that may be obtained under certain conditions by the use of a berm. Berms are not cure-alls, but, when used judiciously, they can save a great deal of material and still ensure a safe structure.

To illustrate the choice of type of structure to use in the design of shore protective works, the following example is given. Utilizing the analyses of the results of the Waterways Experiment Station data shown in Fig. 12 and Eqs. (21) and (27), the wave up-rush has been computed for three berm-type structures, a composite slope structure, and a continuous slope structure. Each structure was designed to prevent overtopping by a twelve-foot-high, six-second-period wave. The storm water level was assumed to be five feet above mean water level. The crown of the structure was designed eleven feet wide and the land slope was 1:2.5. The results of the analyses are shown in Table 4.

The composite slope has the lowest crest elevation but requires the most space. On the other hand, the continuous slope needs the minimum width but has the highest crest elevation. Surprisingly, the composite slope has either the largest or the least cross-sectional area, depending on whether the slopes were 1:3 to 1:20 or 1:20 to 1:3. The actual choice of design depends on the local problem. For instance, quite often local inhabitants do not want a structure that will cut off a pleasant view of the water, in which case the designer might have to resort to the structure with the lowest crest elevation. In this example, the 70-foot-wide berm and the composite slope structures had approximately the same wave up-rush and, therefore, the minimum crest elevation. In other words, a four-foot-high vertical wall at the end of the 70-foot-wide berm had the same effect in reducing wave up-rush as 76 feet of additional 1:20 slope.

It is interesting to note that the steeper continuous slope compared very favorably with the berm-type structures. However, up to this time only smooth impermeable slopes have been discussed. In the next two sections it will be shown that the slope of the structure is very important when considering the dissipation of wave energy caused by roughness and permeability.

Roughness

It is known that the addition of roughness elements to a surface increases the turbulence and that this dissipates energy. Many structures have been built incorporating roughness in an attempt to reduce the wave up-rush—some have been successful and others have not. At the present time, not enough data is available to make quantitative statements, but experience can be drawn upon to give some qualitative hints.

The turbulence caused by the roughness elements transmits shear. The general equation for shear stress is of the type

$$\tau = \rho \, K V^2 \qquad (29)$$

where τ is the shear stress.

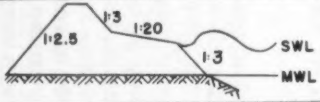
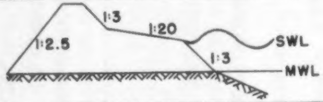
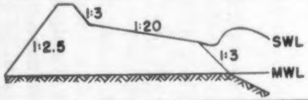
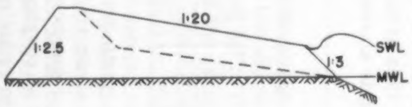
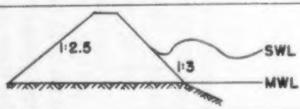
ρ is the density of the liquid

K is a factor depending on the roughness elements, their size and spacing.

V is the velocity near the bottom.

Table 4

Comparison of Types of Design Structures

Structure	Sketch	Wave Data H (Ft)	Wave Data T (Sec)	$\frac{H}{T^2}$	$\frac{R}{H}$	R Feet Above SWL	Crest Elev.	Width	Cross Section Area (Sq Ft)
30 Ft. Berm		12	6	.333	0.89	10.7	15.7	122.8	997
50 Ft. Berm		12	6	.333	0.72	8.6	13.6	128.3	924
70 Ft. Berm		12	6	.333	0.63	7.6	12.6	139.8	977
Composite		12	6	.333	0.61	7.3	12.3	202.8 (1:3 TO 1:20) 163.7 (1:20 TO 1:3)	1625 (1:3 TO 1:20) 764 (1:20 TO 1:3)
Continuous		12	6	.333	1.05	12.6	17.6	107.8	1045

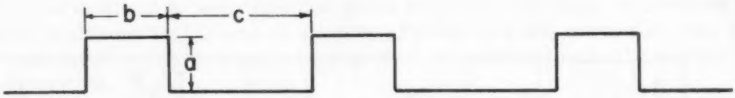


Figure 13. Schematic of rectangular type artificial roughness

Much experimental work has been accomplished to determine the spacing of artificial roughness in the form of blocks which will give the maximum dissipation of energy. With reference to Fig. 13, Johnson(17) has shown that when $\frac{b}{a} = 4$, the maximum roughness coefficient is found when $\frac{c}{a} = 12$.

Very little is known about the ratio of roughness height, a, to the wave height, H_o. Bruun(16) has recommended using roughness so that $\frac{H_o}{a}$ is 3 to 4. It is of greatest importance that the roughness is of a size large enough to be "felt." In the experiments on the North Kent Sea Wall, conducted by the Laboratorie Dauphinois it was found that artificial roughness placed according to the results of Johnson decreased overtopping appreciably. It was proven in this case that small step-type roughness did not decrease the run-up to a great extent. A step-type roughness does not decrease the reflection, although it affects the run-up by the ratio of the step height to the wave height.

Miche(8) has analyzed the velocity of a particle of water along the bottom as an unbroken wave rushes up an infinite incline. He found that

$$V_{max} = \frac{H\ \pi}{\sin \alpha} \sqrt{\frac{g}{L}} \tag{30}$$

To give one an idea of the great increase in velocity as the slope diminishes, the following table shows relative values of V_{max} for a given γ_o.

Table 5

Computed values of V_{max} for given slopes, from Miche[8]

α	90°	45°	30°	18°	15°	10°	5°	2°	1°
V_{max}	1	2	3.46	7.25	9.45	17.2	48.5	192	542

There will be a much greater dissipation of energy by roughness on a gentle slope than on a steep slope because of the great increase in the water particle velocities and, if roughness is to be used, it is much more effective on a gentle slope. The use of roughness on a berm of sufficient width would undoubtedly be of great aid in reducing up-rush. But, because of the complex nature of the problem, it is highly recommended that any design incorporating artificial roughness be tested in a hydraulic model. The construction of artificial roughness is expensive and may be of dubious usefulness, especially under storm conditions.

The quantitative effects of small roughness can be obtained by analyzing recent tests conducted at the Beach Erosion Board on beaches of varying types of sand and uniform stone. In these tests, a layer of sand was placed on a concrete incline and wave up-rush tests were made. In this way, although

there was sand roughness, there was no permeability. Considering only breaking waves, where $\frac{H}{T^2} > i^2$, it is found that the wave up-rush can be approximated by the formula

$$\frac{R}{H} = \frac{2.3}{\left(\frac{H}{T^2}\right)^{1/2}} (\tan \alpha)(r) \tag{31}$$

where (r) is a roughness factor.

This equation is equivalent to that for an impermeable smooth slope multiplied by a roughness factor (r). Table 6 gives the sand size, slope, and corresponding values of (r) which were determined from the Beach Erosion Board tests.

Table 6

Roughness factors for varying sand sizes and beach slopes, computed from data of the Beach Erosion Board

Sand Size	(r) Slope 1:10	(r) Slope 1:30
Smooth	1.00	1.00
0.2 mm	0.96	0.89
1.0 mm	0.85	0.78
2.0 mm	0.82	0.71
3.4 mm	0.76	0.64
Blue Stone	0.70	0.49

From the results given in Table 6, it is obvious that the slope of a protective structure or a beach plays an important part in the effect of roughness. Sibul(15) has used an artificial roughness with a Manning (n) of 0.13 on slopes of 1:2 and 1:3. Although this roughness greatly exceeds the roughness of blue stone, the (r) factor can be computed to be consistently about 0.80 for both slopes. Thus, roughness on steeper slopes has less effect on the wave uprush. This statement is somewhat in variance with the opinion of Bruun,(16) but it can be further substantiated from the information contained in an excellent study of the <u>Reflection of Solitary Waves</u>(18) by Caldwell. Plate 13 of his study shows that the energy absorbed by a permeable structure varies twofold for different sizes of rock in a vertical wall; but, at a slope of 15°, the energy absorbed is the same even though the median diameter of rock in the structure varies sixteen times. In this test, the porosity of each size of rock was the same, however, the energy absorbed increased greatly with decreasing slope and varied somewhat with the rock size.

Actually, the only types of structures that have roughness and not permeability are impermeable sea walls with concrete block or concrete step-type elements. To facilitate the design of such structures, extensive research is necessary to compute the roughness factors for the various types of roughness elements.

Permeability

This brings us to the last major method by which the energy of an oncoming wave is dissipated—heat generated due to the mixing in the voids of a permeable structure. The author has concluded that permeability plays a much more vital role in dissipating energy than roughness does. Although it is possible to have roughness without permeability, it is not possible to have any degree of permeability in a protective structure without some roughness.

Laboratory investigations of wave up-rush show a tremendous reduction in run-up when porous structures are used. This can be seen in the work of Granthem.(12) The work of Caldwell(18) is also interesting in this respect, although it is concerned with reflection of a solitary wave. Caldwell shows that the per cent of wave energy absorbed by the structure varies linearly with the porosity up to values of porosity of 50 per cent, at which time, 90 per cent of the wave energy is absorbed. He also shows that the thickness of the structure is of importance in the absorption of wave energy. For a vertical permeable sea wall, he finds that there is no further absorption of incident wave energy for values of $\frac{W}{D} > 2$, where $\frac{W}{D}$ is the wall thickness divided by the water depth.

The reduction in wave up-rush attributed to roughness and permeability can also be determined from the tests of the Beach Erosion Board. As previously explained, the effect of roughness alone was obtained by placing granular beach material in a thin layer on a concrete slope. The effect of roughness and permeability together was obtained by eliminating the concrete base and constructing the whole slope of the granular material. The author found that the ratio of wave up-rush to wave height for waves breaking on a rough and porous continuous slope could again be determined by the general-type formula

$$\frac{R}{H} = \frac{2.3}{\left(\frac{H}{T^2}\right)^{1/2}} \tan \alpha \ (r) \ (p) \tag{32}$$

where (r), the roughness factor, indicates the effect of roughness and (p), the porosity factor, indicates the effect of permeability.

The fact that in Eqs. (31) and (32) the ratio of wave up-rush to wave height varies directly with the roughness factor (r) or the roughness factor and the porosity factor (r)(p) appears to over-simplify a complex phenomena. However, actual results indicate that this is the case, as can be seen in Fig. 14, where the experimental data follows very closely the curve

$$\frac{R}{H} = \frac{2.3}{\left(\frac{H}{T^2}\right)^{1/2}} \left(\frac{1}{10}\right) \ (.74) \tag{33}$$

I have calculated the reduction of wave up-rush attributed to roughness and porosity from the Beach Erosion Board data and listed the values of the factors (r) and (p) in Table 7.

Note the great effect of permeability when blue stone is used. It is unfortunate that I do not have the actual porosities of the materials used, but it was visually apparent that the blue stone had a much greater porosity than the sand. The run-up was reduced 54 per cent due to permeability alone.

From the one value available to me on the 1:30 slope, it appears that the reduction of wave up-rush due to permeability is greater on lesser slopes. This is to be expected.

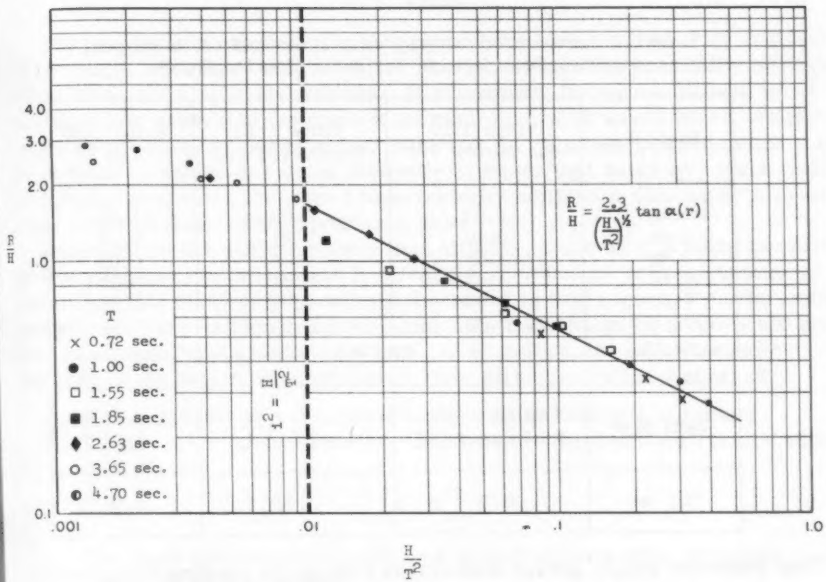


Figure 14. The ratio of wave up-rush to wave heights for varying values of $\frac{H}{T^2}$ for a continuous rough permeable beach slope of 1:10, data from Beach Erosion Board

There is no doubt that a porous structure greatly reduces wave up-rush. However, extreme caution should be exercised when considering the use of porous material, because such material is subject to wave attack and the uplift pressures caused by receding waves. In Holland, the author has seen huge rocks which had been lifted out of place by uplift pressures. Again, in England, the author has seen the force of a wave striking a rubble sea wall tear it apart. The stability of a rubblemound breakwater or porous sea wall should be tested in the laboratory.

It may be many years before systematic laboratory investigations will give conclusive quantitative results concerning the actual effects of various types of construction material on wave up-rush. As previously stated, it is unusual—except in concrete block or step-type artificial roughness—to find a roughened structure without porosity. Therefore, the roughness factor (r) and the porosity factor (p) can be combined into a single reduction factor (ϕ) where

$$\phi = (r)(p) \tag{34}$$

The ratio $\frac{R}{H}$ theoretically should be a function of $\tan \alpha \left(\frac{H}{T^2}\right)^{-1/2}$. The general formula for computing the wave up-rush on a porous, rough continuous slope is therefore

$$\frac{R}{H} = 2.3 \frac{\tan \alpha}{\left(\frac{H}{T^2}\right)^{1/2}} (\phi) \tag{35}$$

Table 7

Porosity factors for varying sand sizes and beach slopes, computed from data of the Beach Erosion Board

Sand Size	Slope 1:10 Non-porous (r)	Slope 1:10 Porous (r) (p)	Slope 1:10 (p)
Smooth	1.00	1.00	1.00
0.2 mm	0.96	0.91	0.95
1.0 mm	0.85	0.82	0.96
2.0 mm	0.82	0.74	0.90
3.4 mm	0.76	0.65	0.85
Blue Stone	0.70	0.38	0.54

Sand Size	Slope 1:30 Non-porous (r)	Slope 1:30 Porous (r) (p)	Slope 1:30 (p)
0.2 mm	0.89	0.77	0.86

Thus whenever rough, porous material is used in the construction of shore protective works, the reduction factor ϕ will measure the effectiveness of the material in dissipating the oncoming wave energy. The most effective material is that which has the least reduction factor. Table 7 shows that the reduction factor for blue stone on a 1:10 slope is 0:38. This demonstrates that blue stone on the 1:10 slope reduced the wave up-rush 62 per cent over what it would have been on a smooth impermeable slope; consequently it was highly effective. The results of laboratory tests indicate that the up-rush of surging waves is reduced by roughness and permeability in the same ratio as the uprush of breaking waves; that is, by the reduction factor, ϕ.

In the past few years, the tetrapod design of the Laboratorie Dauphinois has become very popular in Europe for harbor breakwaters. The tetrapods present a very rough surface and have a porosity, according to Waterways Experiment Station, of approximately 50 per cent. They can be constructed in any size, and, therefore, in any weight, and because of their interlocking qualities are very stable.

An example was given in Table 4 illustrating the types of structures which can be used in the design of shore protective works. It was seen that a 1:3 continuous slope structure compared favorably with a 70-foot-wide berm-type structure when both structures had impermeable slopes. From the discussions of the effects of roughness and permeability on the dissipation of energy, it should be realized that not only will an appreciable reduction in uprush occur with the use of rough and permeable structures—thus conserving space and material—but that the effects of roughness and permeability are most pronounced in reducing wave up-rush on gentle sloping structures. Therefore, it is doubtful if the 1:3 continuous slope structure would have compared favorably with a berm-type structure that judiciously employed roughness and porosity.

SUMMARY

The conclusions expressed in this paper have been arrived at by analyses of hydraulic laboratory model tests. There is always the possibility of scale effect when utilizing model results. Furthermore, the waves utilized were mechanically generated, as opposed to wind-generated waves which one encounters in nature. Consequently, there was no variation in wave attack such as is found in nature due to the diversity of waves that make up a wave train. For design purposes, the wave characteristics utilized in this paper may be considered as those of the significant wave.

Research on the subject matter is continuing; much of it is being conducted with wind-generated waves and large-scale wave tanks. It is the opinion of the author that many of the concepts discussed in this paper are fundamental and will not vary—although derived constants may change in large-scale model studies or with wind-generated waves. In summary, the following salient features of the design of coastal protective structures were discussed.

1. If at all possible, wave reflection should be kept to a minimum.
2. To minimize wave reflection, the slope of a protective structure should be such that

$$i^2 < \frac{H}{T^2}$$

3. The wave up-rush on a continuous sloped impermeable structure can be computed by the equation

$$\frac{R}{H} = \frac{2.3 \tan\alpha}{\left(\frac{H}{T^2}\right)^{1/2}} \qquad \left\{\begin{array}{l} i^2 < \frac{H}{T^2} \\ H \approx H_o \end{array}\right. \tag{21}$$

4. For surging storm waves, the wave up-rush to be used for design purposes can be approximated by

$$\frac{R}{H} \approx 3 \qquad \left\{\begin{array}{l} i^2 > \frac{H}{T^2} \\ H \approx H_o \end{array}\right. \tag{22}$$

5. The wave up-rush on composite slopes where the storm water is at the break in slope can be computed by the equation

$$\frac{R}{H} = \frac{2.3}{\left(\frac{H}{T^2}\right)^{1/2}} \left(\frac{\tan\alpha_1 + \tan\alpha_2}{2}\right) S \qquad \left\{\begin{array}{l} i_1^2 < \frac{H}{T^2} \\ H \approx H_o \\ S \approx .8 \text{ to } .9 \end{array}\right. \tag{27}$$

6. Berms will reduce wave up-rush. The berm width should be such that

$$\frac{B}{L} \geq \frac{1}{5} \tag{28}$$

The use of a vertical wall at the termination of the berm has proven satisfactory in dissipating energy.

7. Artificial roughness is much more effective on gentle slopes than on steep slopes. The roughness reduces the wave up-rush on a continuous sloping structure according to the equation

$$\frac{R}{H} = \frac{2.3}{\left(\frac{H}{T^2}\right)^{1/2}} \tan\alpha(r) \qquad \left\{\begin{array}{l} i^2 < \frac{H}{T^2} \\ H \approx H_o \end{array}\right. \tag{31}$$

8. The general formula for computing the wave up-rush on a porous, rough, continuous slope is

$$\frac{R}{H} = 2.3 \frac{\tan \alpha}{\left(\frac{H}{T^2}\right)^{1/2}} (\phi) \tag{35}$$

where ϕ, the reduction factor, is a measure of the effectiveness of the material in dissipating the oncoming wave energy.

9. Tetrapods, or a similar-type material, are ideal construction material because they incorporate both roughness and porosity, have interlocking qualities, and can be constructed in any size.

Application of Results to Central and Southern Florida Project

The design of the levees at Lake Okeechobee is currently of great interest to the Office of the Chief of Engineers. The author understands that the redesign of the levees contemplates the use of an impervious continuous slope of 1:3 or less or a composite impervious slope of approximately 1:10 to 1:3. The wave steepnesses of storm waves in shallow water are generally greater than one twenty-fifth, $\gamma_o > .04$. The critical slope is defined as

$$i = \sqrt{5.12\ \gamma_o} \tag{10}$$

so that for the design slopes contemplated and the expected wave steepnesses the waves will break upon the levees. Therefore, the wave up-rush can be computed by either Eq. (21) or Eq. (27).

Fig. 15, which has been taken from the publication "Waves and Wind Tides in Shallow Lakes and Reservoirs"(3) shows that a very good approximation of the relationship between the significant period and the significant wave height can be obtained by the parabola

$$\frac{H}{T^2} = \frac{1}{4.8} \tag{36}$$

Substitution of the above equation into Eq. (21) gives

$$\frac{R}{H} = 5 \tan \alpha \tag{37}$$

Substitution of Eq. (36) into Eq. (27), using an S value of 0.9 gives

$$\frac{R}{H} = 4.5 \left(\frac{\tan \alpha_1 + \tan \alpha_2}{2}\right) \tag{38}$$

The author believes that the values of wave up-rush computed by the use of Eqs. (37) and (38) can be safely used as a basis in the redesign of the levees at Lake Okeechobee. A reduction factor, ϕ, should be applied to the values obtained from Eqs. (37) and (38) if other than impervious slopes are considered.

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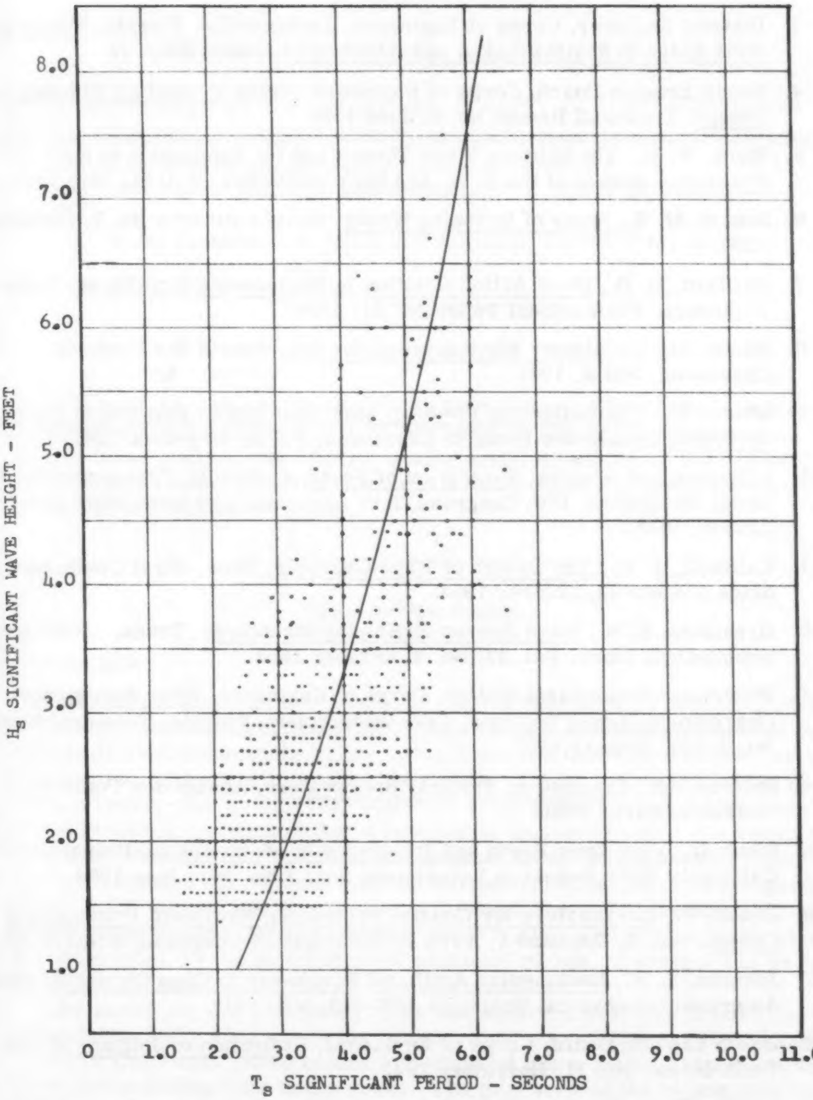

Figure 15. Relation between significant wave height and significant wave period, Lake Okeechobee, reproduced from Reference 2, Figure 25

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AN ELECTRIC ANALOG MODEL OF A TIDAL ESTUARY

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ABSTRACT

Five hundred miles of channel in the Delta Region, California, now subject to sea water encroachment, may be protected by salinity barriers and a master levee system. An electric analog model developed at the University of California predicts tidal amplitude and flows resulting from such modifications to the hydraulic system.

Part I—The Problem

Introduction

The Delta Region, located about sixty miles east of San Francisco, occupies a strategic position in the geography of California, since it lies between the relatively arid San Joaquin Valley to the South and the Sacramento River Valley to the north, which has a developable surplus of water. The California Water Plan, a twenty-five to fifty year blueprint for the development of the state's water resources, envisions that a substantial amount of water will eventually be transported southward across this general area. At the present time, the Tracy Pumping Plant, operated by the Bureau of Reclamation, is designed to divert 4,500 second feet of water from the southern rim of the Region to the area west of Fresno, about a hundred miles further south. Eventually this rate will be tripled, by the installation of additional plants, to serve areas as far away as Los Angeles.

The entire region, embracing some 730 square miles, was at one time an extensive marshy area where the two principal rivers draining the inland valley of California joined before they emptied into a series of bays and thence through the Golden Gate to the ocean. The peat soils of the region have proved

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very fertile when drained, and practically the entire region has been reclaimed into fifty or so islands, nearly all of which lie about ten feet below sea level. In the process of reclaiming the land, some five hundred miles of sloughs and channels were left, so that at the present time one definitely gets the first impression of a vast maze of waterways lacing the area.

At first sight, this maze of waterways would appear to provide an ideal means of bringing additional water southward, with perhaps only local improvements needed to enlarge the channel capacities. A more detailed examination, however, reveals a number of complicating circumstances. One is that a sea level channel to Stockton, on the eastern rim, must be maintained. The natural summer flow of the streams tributary to that region is not sufficient to repel all of the saline water which is moved by tidal mixing action in the channels leading from San Francisco Bay. Furthermore, the summer flow of the San Joaquin River contributes further to the salt content of the area, since it serves as the agricultural sewer of the northern part of the San Joaquin Valley.

These adverse salinity effects are being controlled to a certain extent at the present time as a part of the Bureau of Reclamation operation of the Central Valley Project. Good quality water is released steadily throughout the summer from Shasta Reservoir on the Sacramento River in the northern part of the state. The flow is scheduled to maintain a steady flushing action in the channels leading from the Delta Region westward to the ocean. A flow on the order of 3,500 second feet is required to keep the 1000 parts per million chloride ion concentration point below the confluence of the two rivers near Pittsburg.

In the future, however, this water will be required for higher uses, and it will be necessary to segregate and prevent commingling of the higher quality conserved water from the northern part of the state with the lower quality, higher salinity water that now enters the Delta Region from the sea and from agricultural drainage. The accomplishment of this task will involve major changes in the channel system and consequent alterations in the pattern of the tides in those channels which are still open to the influence of the sea. Although the details of a practical plan for the Delta are still being worked out by engineers of the California Department of Water Resources, several definite features are emerging. These are:

1. Salinity barriers will be placed in the upper parts of the Sacramento River System so that the major part of flood flows from the Sacramento Valley can continue to pass unimpeded from the Yolo bypass through the lower Sacramento, and into Suisun Bay (see map). Surplus water from the northern part of the state, carried by the Sacramento River, will be diverted southward at this point.
2. Diverted fresh water will be passed under the Stockton Deep Water Channel by inverted syphons; the shipping channel will be isolated from the fresh water channels and will be open to the sea. It will continue to serve as a floodway and as a drain for poor quality irrigation return flows and other waste water.
3. Major floodways will be isolated from the secondary channels by a master levee system. This will substantially reduce the number of miles of levee which must be maintained and will remove most of the secondary channels from the influence of tidal action. The latter will continue to be used for the distribution of irrigation water.

From a hydraulic standpoint, these changes are substantial, and normally one would not proceed very far with any plans involving such changes without first making a model study. A particularly cogent reason is that the introduction of barriers in a tidal system can easily result in a resonant amplication of the tide at the barrier and downstream from it. However, in the present instance a reasonably scaled hydraulic model would by very expensive because of its sheer size. At a horizontal scale of a thousand to one, which is the scale used by the Corps of Engineers for the San Francisco Bay model, the area covered would be about four hundred feet square and would equal that of three football fields. Such a model would have to include not only the Delta Region proper, but also the entire San Francisco Bay, because it is necessary that reflected waves due to the tide be allowed to traverse the entire system from the ocean outlet to the farthest upstream area of interest. The model must be constructed with the same scale factors throughout, and represent with the same scale the 4 million c.f.s. peak tidal flow at the Golden Gate and flows as small as one thousand c.f.s. in some of the upstream channels.

It was with this problem and with a severely limited budget of time and money that the State Department of Water Resources approached a staff member at the University of California in Berkeley to see if there was any chance of finding analytic solutions. In a previous study, two members of the University staff had made a series of analytic calculations for the prediction of tidal amplitudes below various proposed barriers in San Francisco Bay proper.(1) However, the geometry of the upstream channels is so complicated by branching that no analytic treatment seems possible. Instead, the staff member suggested that an electric analog model might give some of the answers needed for a relatively small cost, and that some work along these lines was already being done in the Hydraulics Laboratory. After further consultation, a service agreement was made with the University for the construction and operation of an analog of the Delta Region. The agreement period extended from March 1, 1956 to June 30, 1957.

Part II—The Electric Analog Model

What is an Analog Model?

From a fundamental standpoint, one physical system can be considered an analog, or model, of another when it is governed to a satisfactory degree of approximation by the same equations as is its prototype. In this sense, a hydraulic model is an analog of the full scale structure it represents; and furthermore the electric analog models to be described below bear a remarkable resemblance, in their adjustment and operation, to hydraulic models. The term analog models is used to avoid a confusion with analog computers, which are machines for directly solving ordinary differential equations.

From a slightly less fundamental standpoint, it can be said that two systems are analogous when we observe the same kinds of action in each. Thus a very long wire can transmit electric waves along its length in a fashion very similar to the way a long canal will transmit a train of long period waves along its length; furthermore, at the open end of the wire the wave is reflected in the same way as at the closed end of a canal.

Here, as in the analogs to be described later, the voltage level is the analog for water surface elevation and electric current is the analog for discharge rate.

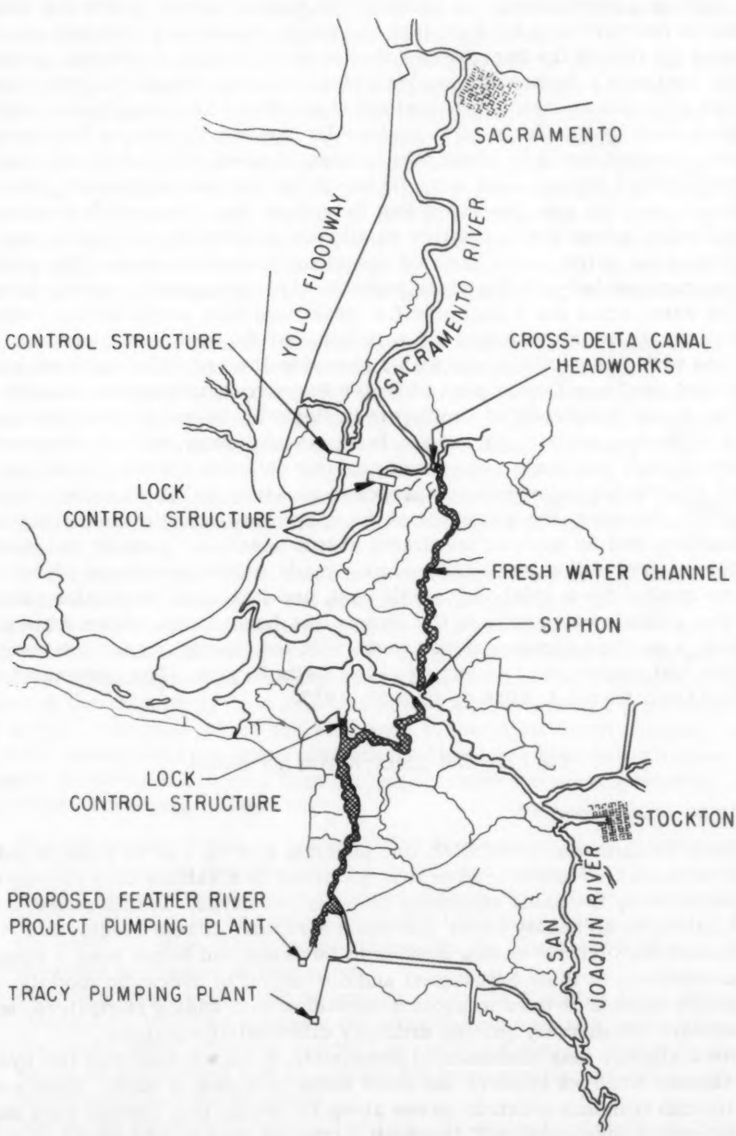

Fig. 1. One of several plans for improvements in the Sacramento-San Joaquin Delta Region (modified Biemond Plan).

In this example the action is the same, but the equations are only the same to a first approximation. In order to make the best use of such an analog for the study of hydraulic systems, the fundamental equations must be examined to see in which ways they are similar, and in which ways they differ. The main difference, in the case of tidal flow analogs, is in the way energy is dissipated; if the electric circuits can be altered in this regard to correspond more closely to the hydraulic system, it can be shown that it is possible to neglect some of the other differences, or to at least make corrections for them.

One difference is that in a canal the momentum depends on the depth as well as the flow rate whereas the corresponding property of an electrical circuit, the strength of a magnetic field, depends only on the electrical current. The practical effect is that a type of non-linearity which distorts the tide-induced waves is not present in the electric analog. Its magnitude depends on the wave-amplitude to depth ratio. However, in those cases where this ratio becomes appreciable, the friction has usually increased to the point where it predominates over even the linear portion of the inertia, so that from a practical standpoint the most important non-linearity for tidal flows and river flood waves is due to friction.

The friction, like the inertia, depends on the water depth. Furthermore, since there is only an electric analog quantity for discharge, and none for velocity, its value must be expressed in terms of the discharge before conversion into electrical terms. If q is the discharge per unit width and y is the water depth, the friction slope is given by an arrangement of the Chezy equation by:

$$s_f = - \frac{|q|q}{C^2 y^3}$$

This shows (by the absolute value notation) that the sign of the slope is always such that the flow is opposed, and that there is a strong dependence on the water depth. The quadratic form, apart from intensifying the friction in regions of high flows, introduces a symmetrical distortion to the waves. Without the dependence on depth, the distortion would affect positive and negative parts of the wave equally, generally "clipping" the maximum flow peaks. However, for progressive waves, an upstream flow is associated with greater depths and ebb flows with smaller depths. Even if the discharge and surface displacement are not exactly in phase (phase is here defined as the relative displacement of the maxima), there will be a net reduction in the friction for upstream flows, while downstream (ebb) flows encounter increased friction. Lacking an exit for flows at the upstream end, a compensating increase in the average water level will occur with distance upstream. This is observed as an increase in the "tidal plane" elevation and is found in estuaries whether there is a super-imposed river flow or not.

The overwhelming importance of the friction contribution to the non-linear characteristics of the hydraulic system led us to develop a special electrical unit which simultaneously simulates the square-law characteristic and the depth dependence of the friction. This battery operated transistor circuit is described in more detail in Appendix I; it operates for current flow in each direction, and is relatively simple. Over twenty were built for incorporation in the Delta Analog.

A third way in which the two systems differ is that the properties of the electrical transmission line must be lumped into discrete elements of inductance, resistance, and capacitance. That is, the river channels must be broken into reaches, each of which are then simulated by a set of electrical elements. If these reaches are too short, there is a waste of effort in building

too many elements; but if they are too long relative to the length of the waves which must be transmitted, the higher frequency components will be attenuated artifically relative to the lower. Of more importance, long before the higher frequencies are seriously attenuated they suffer a phase shift which is not correctly related to that of the lower frequencies. A more extensive treatment of the errors due to the above-mentioned non-linearities may be found in reference 2. The errors due to lumping are treated in Appendix II.

A fourth point is that there is no electrical equivalent of kinetic energy and thus no way to include the $V^2/2g$ term in the equation of motion. The magnitude of this term relative to that of the temporal acceleration is given by the Froude Number, and is thus small in most tidal estuaries. A partial remedy for finding the actual depth at a given point is to compute $V^2/2g$ and to correct the indicated water surface elevation by this amount. This will lead to a change in the cross sectional area, but this introduces only a second order correction to the other terms.

In spite of the above list of differences, it has proved possible to obtain results from the analog closely duplicating the existing flows in the Delta. What is indicated is that the operation of an analog model requires at least as great a knowledge of hydraulics as does the operation of a regular hydraulic model. In the latter most of the above-mentioned difficulties are painlessly looked after by the model itself, but in the case of the analog the operator must undertake this responsibility. He must be able to abstract the properties of the hydraulic system and translate them into electrical terms, and vice-versa. On the whole, however, there is a striking similarity between hydraulic models and analog models, from the standpoint of construction and operation, that lends some familiarity to the latter.

Analog and Hydraulic Models Compared

The steps taken in the Delta Analog Study paralleled in many ways those taken in a more conventional investigation, and perhaps it will be helpful in explaining the newer method to compare it wherever possible with the more familiar. The first step in each case is to gather sufficient information about the prototype channels so that they can be duplicated in the laboratory—in the one instance in the form of miniature channels, and in the other in the form of suitable electric circuit elements.

The technique of constructing a reduced scale model of the prototype channels is straightforward, if somewhat laborious. Usually a considerable amount of bottom detail is included, and perhaps it is only in this way that some details of the flow, such as velocity distributions, can be duplicated. This type of information cannot be obtained from an analog model, which duplicates the wave transmission characteristics of the channels and gives only the velocity averaged over the cross sections.

Preliminary to the construction of the analog model the prototype channels must be divided into reaches within which the cross section does not change too much and which are short enough for the purposes of the investigation. For the investigation of tidal flows, this length is about five miles, and for flood waves it can be twenty-five miles or more. The wave transmission characteristics of this reach depend on the average cross sectional area, the water surface area, and the friction. The first two are obtained from field data, as in the case of hydraulic models, but the friction must sometimes be determined by trial, as is nearly always the case with hydraulic models.

In the construction of the Delta Analog, we have assembled the electrical components for each channel reach into a small plug-in unit. Each contains a variable inductor, a variable resistor for use in introducing linearized friction, and a variable capacitor. The relations between the hydraulic parameters and the values of time scale, capacitance, inductance, resistance, etc., that must be used, are well described elsewhere (see for example reference 3 or 4) so that it is sufficient to remark that the value of capacity is directly proportional to the water surface area, and the inductance is related to the cross sectional area and the length of the reach. The plug-in units are adjusted to the indicated values of inductance and capacitance on a special test panel, and the resistance is set according to the best estimates of the friction. The resistance value is further adjusted in the verification procedure or when square-law friction elements are introduced.

The sockets to receive these units are installed on a large table, and are interconnected by wiring on the under side. After the adjustment of the plug-in units they are inserted in their sockets and the analog is ready for the second stage, that of verification.

In a hydraulic model, the corresponding steps are the physical construction of the model channels. If the model has been built to an exaggerated vertical scale, as is usually the case, the model friction must be increased in the same proportion and this is often done by inserting copper strips into the bottom. In the process of verification, these strips can be bent down, or straightened up, until the tides of some historical period are satisfactorily duplicated. During the verification stage of the operation of an analog model, the friction is adjusted in a similar way, except that the adjustment is done with a screwdriver on the individual units. At this point the hydraulic model is in most instances ready for the active phase of the investigation, but one additional verification step is usually necessary in the case of the analog, to insure that the correct cross sectional areas have been introduced. The inductance value should be proportional to the length divided by the average cross sectional area, or more correctly the square of the length divided by the total volume of water in the reach. The part of the volume contained in shoal areas that does not contribute appreciably to the inertia, because of its low velocity and isolation from the main stream, should be excluded from the determination. Although it is not always possible to determine at first the correct amount to exclude, if the correct value is chosen, the velocity of wave propagation in the prototype channel will be duplicated in the analog.

The results of one verification run are given on the next page. The amplitude (peak to trough) and phase (time lag) of the tide in the channels leading from the Golden Gate to Sacramento, a distance of one hundred miles, are shown by the circled points. These measurements were made on the neap tides of September 13, 14, 1954, for which good records are available. The results from the analog are marked with squares.

Tidal amplitude could be measured to 0.05 foot on the analog and time to the nearest 10 minutes. The field data are determined to about the same accuracy; nearly all of the discrepancies are explained in this way. The analog measurements of tidal plane elevations were too low by 0.25 foot at the junction of the two rivers; this is accounted for by the salinity gradient. We could find no correlation between the daily variations of the tidal plane there and wind direction and velocity at Berkeley, thirty miles to the southwest.

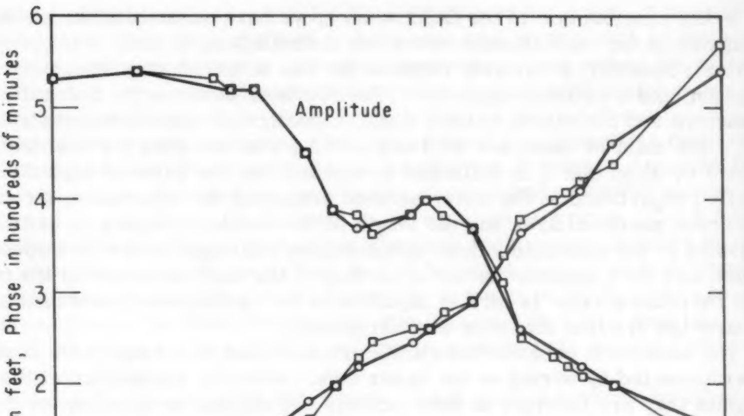

Fig. 2. Tidal amplitude and phase in the Sacramento River from the Golden Gate to the city of Sacramento. Circles represent prototype data; squares represent results from the model.

Tide Generation and Measurement

Both neap tides and spring tides were simulated; the corresponding voltage waveforms were generated at low power level and then introduced at a point corresponding to the bar outside of the Golden Gate by a low output impedence amplifier. The neap tides were represented by a 5000 cycle per second sine wave, and the spring tides were generated with a photo follower - masked cathode ray tube device. There does not seem to be very much similarity to hydraulic models here, but the combination of float actuated pumps and cam operated surface elevation controls used in hydraulic models is actually a servo-mechanism with feedback, and corresponds in a theoretical way to our use of heavy feedback to insure a low output impedence for the tide waveform amplifier.

In other ways, too, there are theoretical similarities. When it is desired to terminate a hydraulic model at an upstream point, an artificial impedence is introduced in the form of some kind of energy dissipator plus basin storage. Similar combinations of resistive and reactive impedence can be used in an analog model for the same purpose, but with considerably greater ease.

The electronic equipment for measuring amplitudes, flows, and time intervals in the analog were specially designed. All links in the measuring chain were direct-coupled, allowing the steady state value of voltage or current to be measured together with the alternating, or tidal, fluctuations. Thus a zero level could be fixed on the cathode ray tube display and the absolute value of the flow determined.

Two probes were used in obtaining information; each could be switched to measure either current or voltage, and further switching allowed any two of these four quantities to be displayed simultaneously (except that no provision was made for simultaneous current measurements). Furthermore, by the use of a calibrated delay line, the phase difference or time lag between the two measurements could be determined.

The two selected signals were combined in an electronic chopper and appeared together on an oscilloscope screen, where amplitudes could be scaled from the graticule. The system was calibrated frequently by applying known voltages (from secondary standard mercurous chloride cells) to the probes; thus the accuracy was not dependent on the gain stability of the electronic system, though this was surprisingly good.

Results

Since the Department of Water Resources has not completed their Salinity Barrier investigation, it would be premature to indicate at this place the actual results of the study, as well as their engineering significance. It may be most interesting to the reader on the other hand to see what kind of results may be obtained.

One of the most important questions to be answered by the analog is that of predicting tidal elevations if the channel system is changed by barriers placed into various channel sections, by cut-offs between channels, by enlargement of individual channel sections or by any combination of such changes. Simultaneously, the flow velocities are obtained from which it is hoped that the extent of the expected salinity intrusions may be derived. It has become very clear from the past results that a barrier does not always reduce the tidal prism, but often may cause the tidal flow to be reflected back into the channel system, locally increasing the tidal heights. Such reflections must be reduced to a minimum by the proper location of the barriers.

Similarly, the opening of channels may increase the tidal effects in some places, reducing them simultaneously in other sections. The analog is here an extremely valuable tool in balancing these effects such that undesirable increases of tidal heights may be prevented both in the final plan and in the temporary stages of construction.

At the present time the analog has been moved to Sacramento, where the State Department of Water Resources plans to use it in a continuing study of fresh water flow distribution and to schedule the construction so that closures can be made in the easiest way.

APPENDIX I

The Transistorized Square-Law Resistor

During the investigation, a new type of square-law resistor was developed which should allow a considerable advance in the technique of applying electric

analogs to open channel flow problems. Some of its features are listed here:

1. For a given voltage drop, the current can be adjusted over a range in excess of twenty to one. This allows the same type of unit to be used to simulate large and small channels.
2. For each setting, the voltage drop bears a square-law relationship to the current except for small values of current. This simulates the variation of energy loss with discharge in open channels.
3. For a given voltage drop, the current can also be made a function of the voltage existing between the unit and ground. This simulates the variation of discharge, for a given water surface slope, with water depth.
4. The units are small, operate indefinitely from self-contained batteries, and are low in cost.

Theory of Operation

For steady flow, the discharge in an open channel, according to the Manning formula, is related to the water surface slope, S, the cross sectional area A, and the hydraulic radius R, in the following way:

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2} \tag{1}$$

wherein n is the Manning roughness coefficient. Assuming n to be constant for a given channel section, the factors $\frac{1.486}{n} AR^{2/3}$ can be represented as a function of the elevation z above a horizontal datum; the equation can then be written as:

$$Q = F(Z)\, S^{1/2} \tag{2}$$

Similarly, the magnitude of current, i, in the square law unit, can be expressed by the following formula relating it to the voltage e above ground and the voltage drop, Δe, across the unit:

$$i = f(e)\, (\Delta e)^{1/2} \tag{3}$$

These relationships can be shown graphically in two ways. For a given value of slope S, the discharge - water surface elevation relationship will take

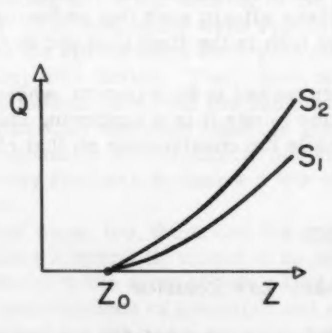

Fig. I - 1

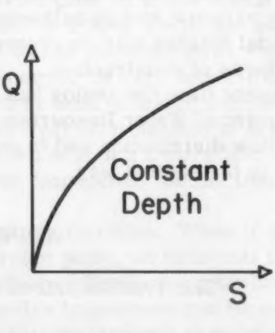

Fig. I - 2

the general form shown in Fig. I - 1, where z_o is the elevation of the channel bottom and S_2 and S_1 are two different values of the slope. Alternatively, for a particular elevation, the discharge-slope relationship takes the form shown in Fig. I - 2. The function F can be approximated, over a limited range, by a power function of z; the exponent is about 1.5 for many natural channels. The corresponding function f depends on the characteristics of the transistor used in the circuit (to be described below), but in general, its approximate power function has an exponent somewhat smaller than 1.5. This is usually of little importance, however, for there is an adjustment available for the slope of the curve and the magnitude of the current at the operating point. Together they can be used to superimpose the curve representing f on that representing F over a reasonable range when the ordinate and abcissa scales have been suitably altered according to the scale factors relating i to Q and e to z. This is shown by the dotted line in Fig. I - 3.

The Circuit of the Square-Law Resistor

The basic characteristics of the square-law resistor depend on those of the transistor employed. Although all transistors have somewhat similar characteristics, discussion will be centered on those of Sylvania type 2N229, which has proved to be entirely adequate and, in addition, to be the lowest-cost transistor on the market at the user net price of $0.75. This transistor is of the type n.p.n., in which the electrons flow from the emitter towards the collector. A third electrode, the base, receives a small percentage of this current. These currents and the transistor nomenclature are shown in Fig. I - 4.

When an external voltage, Δe, is applied between the terminals such that B is more positive than A, a current, i_c, will flow from the emitter towards the collector. Its dependence on Δe is similar to the dependence of Q on slope shown in Fig. I - 2. The current, i_b, shown as being induced by the battery and limited by the resistor r_1, is only about 4% of i_c, but it has a pronounced affect on the latter. Actually, for a given value of Δe, the current i_c can be varied over a range of fifty to one by varying i_b over a similar range, though at a lower magnitude. It should be noted that in this arrangement, the current through resistor r_1 merely circulates and does not flow in any external circuit.

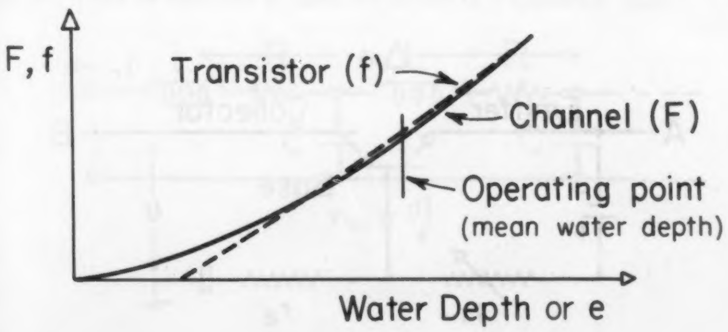

Fig. I - 3

If a resistor r_2 is connected between the base and ground as shown in Fig. I - 4, an additional current will flow out from the base towards ground if the voltage e at the emitter is negative. As before, the current i_c will be controlled by the base current which now depends on e and r_2 as well as on the battery and r_1. The condenser is practically a short circuit at the frequencies used. Now it can be seen that by adjusting r_1 and r_2 and holding Δe fixed, the current i_c can be made to depend relatively more or less on changes in e or on the fixed current supplied by the battery. Practically, this means that both the slope and the current values can be set at the operating point shown in Fig. I - 3. In any of these adjustments, it is almost imperative to be able to examine the characteristic curves in their entirety. This can be done with a transistor characteristic curve tracer (made commercially) or with a circuit described in reference 2.

APPENDIX II

The Effect of Lumping Channel Properties

The important effects of lumping channel properties are shown by taking only the linear properties into account. Then the equivalent electrical circuit can be used together with well-known relationships from electrical engineering.

An artificial, or lumped, transmission line consists of a series of inductance-capacitance elements in the following configuration (resistance is not often large in those instances of interest to electrical engineers, but it is included here because of its increased importance in analog circuits). A progressive wave with a frequency of ω radians per second is modified in traversing the line according to the following expression, wherein e_o is the waveform (as a function of time) at x = 0

$$e = e_o \epsilon^{-\theta x}$$

If the transfer constant θ is a pure imaginary of the form jn, n is the phase shift in radians per unit length of line and there is no dissipation. In terms of the inductance and capacitance per unit length of line, n has the value $\omega\sqrt{LC}$. Where resistance is important, it is introduced through the dissipation factor $d_s = R/\omega L$.

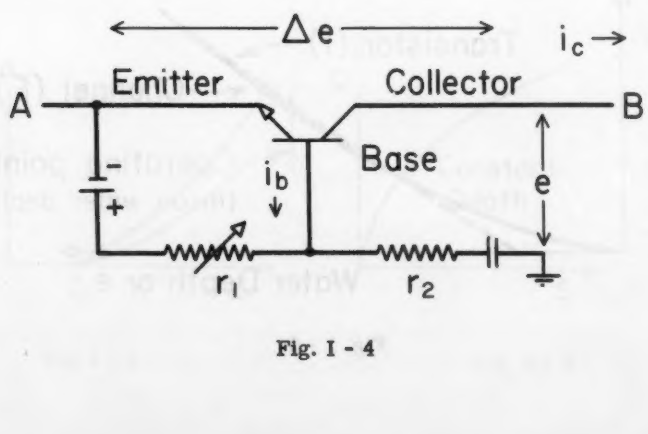

Fig. I - 4

For a single element, occupying a length ΔX, the transfer constant is:

$$\Delta\theta = \tanh^{-1} j n_1 \sqrt{\frac{1 - j d_s}{1 - n^2 (1 - j d_s)}} \tag{2}$$

Here the values of L, C, and R are those of this element and $n_1 = \omega \sqrt{L_1 C_1}$ etc. If the number of elements per unit length is increased without limit and at the same time the values of L, C, and R per element are reduced in proportion, n approaches zero while the transfer constant per unit length becomes:

$$\theta = j n \sqrt{1 - j d_s} \tag{3}$$

This is the correct expression for a distributed-constant transmission line, the analog of a hydraulic channel.

Returning to (2), if n_1 is small, but not zero, an approximation can be made by expanding the inverse hyperbolic tangent in an infinite series and then expanding each binomial with the binomial theorum. If terms on the order of n_1^5 are neglected, the result is:

$$\Delta\theta = j n_1 (1 - j d_s)^{1/2} \left[1 + \frac{n_1^2}{6} (1 - j d_s)\right] \tag{4}$$

Comparing (3) with (4), we see that the effect of a finite n_1 is given by the second term in the bracket. Here $\Delta\theta$ is the transfer constant per element and $n_1 = \omega\sqrt{L_1 C_1}$ the value for this element. If there are k elements in a "wavelength" such that $k\Delta\theta = jkn_1 = 2\pi$, it can be seen that to a first approximation (neglecting d_s) n_1 is $2\pi/k$. Thus if k = 8, such that there are 8 sections per wavelength, the error introduced (for zero dissipation) is:

$$\frac{n_1^2}{6} = \frac{1}{6}\left(\frac{2\pi}{8}\right)^2 = 0.103 \tag{5}$$

If the line is constructed with 8 elements per wavelength, the plus 10.3% error can be compensated for by reducing the values of L, C, and R by the same amount; but if the signal contains harmonics, they will be affected differently because of the frequency dependence of n, and the wave will be distorted due to the lumping process. Using expression 5 for the highest harmonic of interest, k can be selected to limit the error to a tolerable value.

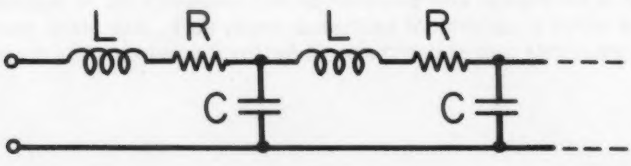

Fig. II - 1

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BREAKWATER AT CRESCENT CITY, CALIFORNIA

John E. Deignan,[1] F. ASCE

ABSTRACT

The outer breakwater at Crescent City, completed in 1957, used tetrapods as the armor stone for the last section. Design of the breakwater and placement and construction of tetrapods provided engineering and construction problems of an unusual nature since this was the first use of tetrapods in the United States. The storm of April 1958 approached design standards; only minor settling of tetrapods was noted.

INTRODUCTION

The outer breakwater at Crescent City extends from a point on the westerly side of the city 3,670 feet southeasterly and then continues easterly for another 1,000 feet for a total length of 4,670 feet. (Fig. 1). The difficulties encountered in constructing the breakwater and the conditions leading to the use of tetrapods for construction of a part of the "dogleg" section have been described in a previous paper in 1954. (1) In that paper, it was pointed out that the average size armor stone required to complete the breakwater was twelve tons. An increasing shortage of this size stone made it necessary to investigate the possibilities of alternatives for armor stone. The last 560 feet of the "dogleg" section of the Crescent City breakwater was completed in October 1957 utilizing tetrapods. This paper describes the design criteria and some of the methods and problems involved in constructing this structure.

Note: Discussion open until February 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2174 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. WW 3, September, 1959.

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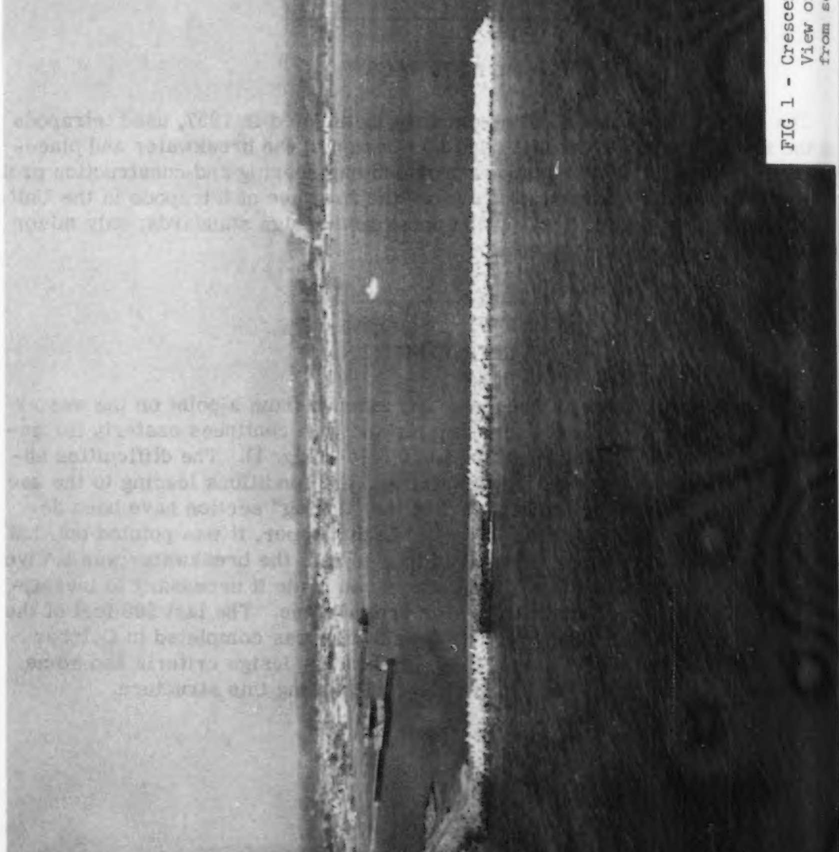


FIG 1 - Crescent City, Calif., 1957
View of completed breakwater
from seaward side

Design

The tetrapod was developed by the NEYPRIC Hydraulic Laboratories in Grenoble, France. It may be described as an integrally cast 4-arm concrete block, any two arms of which form an angle equal to that formed by any other two arms. The arms are shaped as truncated cones and are joined to a central core. (Fig. 2).

Model tests by NEYPRIC Laboratories of the effect of waves on artificial blocks show that one of the most destructive forces to which maritime structures are subjected is a hydrostatic uplift pressure developed within the struc- when a wave recedes from it. Also it has been demonstrated that the destructive action of this uplift pressure could be greatly reduced if the structure was designed to permit a rapid drainage of the water from under the facing blocks. The tetrapod shape was found to be effective in this respect because the high degree of permeability of a mound composed of these units permits a rapid equalization of hydrostatic pressure within the mound and that outside of the structure.

Tetrapod facings also provide a high degree of roughness and tend to dissipate the wave energy into turbulent flow paths which oppose each other in the interstices of the facing. The reduction in wave energy in turn reduces the amount of overtopping and wave reflection from the structure. The shape of the tetrapod is such that adjacent blocks key into one another and with their low centers of gravity provides a very stable facing on relatively steep slopes. Use of steep slopes permits a reduction in the quantity of rock fill required for the core of the structure and also requires less concrete than most other types of facing blocks.

A license was granted to the United States by the NEYPRIC Laboratories to use tetrapods at Crescent City. Prior to granting this license, the NEYPRIC Laboratories suggested that one of their engineers, Mr. Marcel H. Marty, visit the United States to discuss basic principles used by the French in breakwater design using tetrapods.

In using tetrapods for breakwater construction Mr. Marty indicated that the following design criteria had been generally adopted:

a. The weight of tetrapods used is dependent upon the design wave to which they will be subjected.

b. The seaward slope used is 1 on 1-1/3 regardless of the tetrapod size, and two layers of tetrapods are normally used.

c. The intermediate stone layers on the ocean side between the core and the tetrapods are designed to be as pervious as possible. The minimum weight of stone placed immediately below the tetrapods should be one-tenth of the weight of the tetrapods. The minimum stone weight in the succeeding lower layers should be one-twentieth of the weight of the stone of the layer above.

d. Tetrapods generally are not placed on the crest or on the harbor slope.

e. Tetrapods require a backing such as a concrete cap or cut masonry stone to prevent ravelling.

f. NEYPRIC assumes 50 percent voids in determining the number of tetrapods required on the ocean slope.

g. Tetrapods should be founded on a stone mattress if the bottom is sand; if the bottom is rock, the tetrapods may extend to the bottom.

It was emphasized by Mr. Marty that each breakwater must be designed on an individual basis in which all factors and problems applicable to the particular case are investigated.

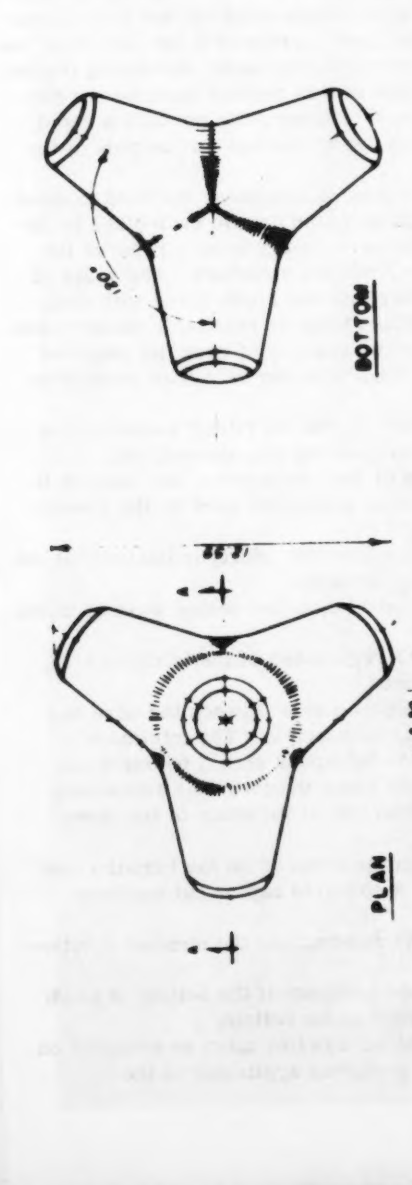


FIG 2 25 TON TETRAPOD DETAILS

Because the Waterways Experiment Station of the Corps of Engineers, located at Vicksburg, Mississippi, has conducted numerous experiments to determine the relationship between the size of tetrapods, slope of breakwater, and other factors pertinent to breakwater design, Mr. R. Y. Hudson, of the Experiment Station, participated in the design discussion and presented the current findings from model analysis. (2) Between the criteria suggested by the French and the experimental data gathered by the Waterways Experiment Station, a design was worked out for the extension of the breakwater at Crescent City.

The design wave height was developed utilizing data from hindcast analyses by the Scripps Institution of Oceanography of wave conditions at on offshore station located approximately 80 miles northwest of Crescent City, and wave observations at the outer breakwater. During the three-year period 1936-1938 short period waves from the southwest ranging from 15 to 20 feet in height occurred at average frequency of .53 days per year. Recent storm wave measurements have substantiated this frequency and range. Wave refraction diagrams show wave convergence at the breakwater, indicating a need for a higher design wave. In view of this data, plus the high cost of mobilizing construction equipment at Crescent City, a 23-foot design wave was selected for the tetrapod section of the breakwater. (Fig. 3).

A preliminary design of the breakwater, using model data developed by the Waterways Experiment Station, indicated that a 15-ton tetrapod on a 1 on 2 seaward slope would probably meet all the criteria for the 23 feet design wave with the exception of that at the end section. Because the tetrapods at the end section would not be backed up by a concrete cap, a 25-ton tetrapod on a 1 on 1-1/3 slope was proposed. In this area the ravelling has to be resisted by the weight of the tetrapod and the interlocking action. Since preliminary cost estimates indicated that a 25-ton tetrapod section constructed on a 1 on 1-1/3 slope would be less than the 15-ton section on a flatter slope, the 25-ton tetrapod design was adopted for the entire length.

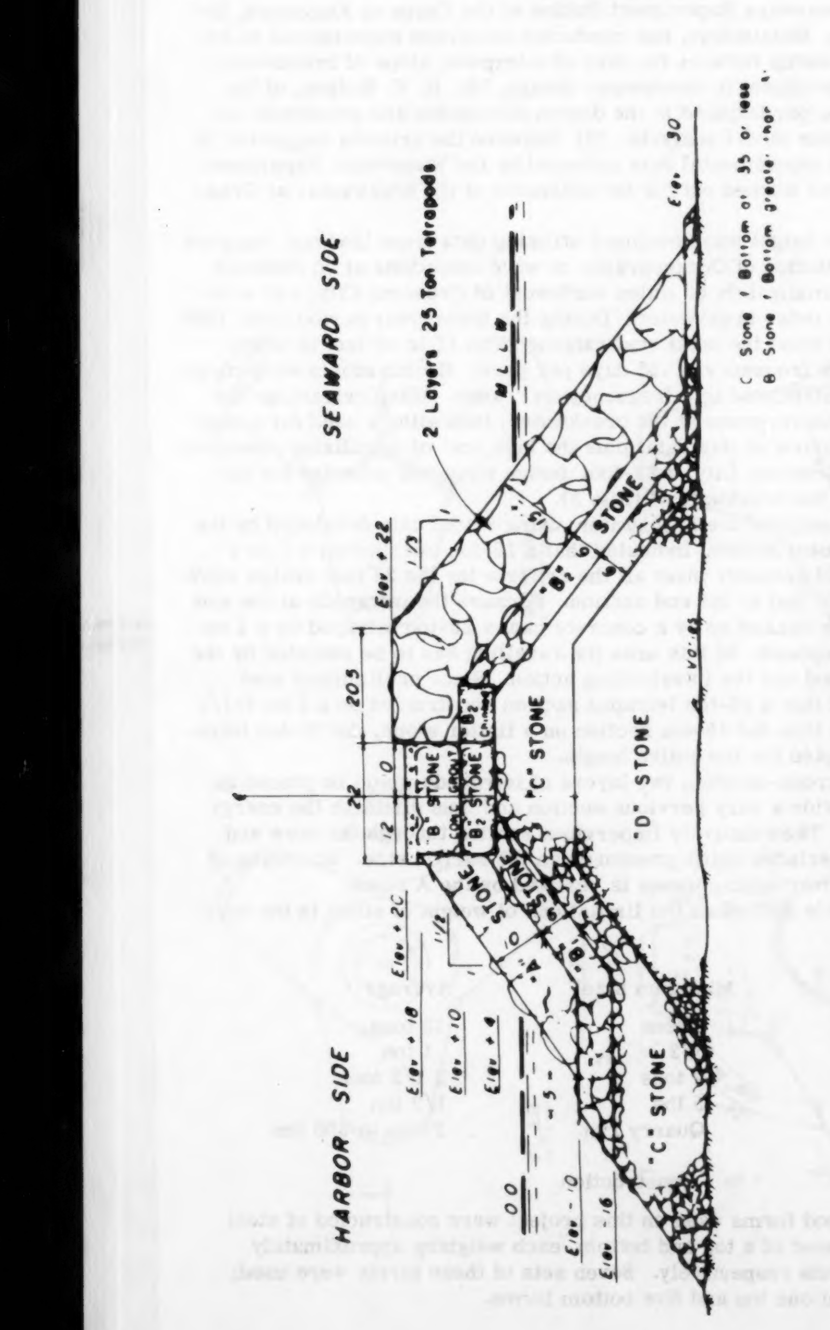
As noted in the cross-section, two layers of tetrapods would be placed on the B_2 stone to provide a very pervious section and thus diminish the energy of the wave action. The relatively impervious section through the core and the concrete cap precludes uplift pressures on the harbor side. Ravelling of the harbor side by overtopping waves is resisted by the A stone.

The following table describes the limitations of weight of stone in the various categories:

Type	Minimum size	Average
A	7 tons	12 tons
B_1	1/2	1 ton
B_2	2 tons	2 1/2 tons
C	5 lbs	1/2 ton
D	Quarry run	Fines to 500 lbs

Construction

Tetrapod forms used on this project were constructed of steel in two pieces composed of a top and bottom, each weighing approximately 3,400 and 3,200 pounds respectively. Seven sets of these forms were used; each set consisted of one top and five bottom forms.



TYPICAL TETRAPOD CROSS SECTION

OUTER BREAKWATER

STA 41 + 20 TO STA 46 + 70

FIG 3

The tetrapods contained approximately 12-1/3 cubic yards of concrete and requires a 9-man crew approximately three quarters of an hour to pour. Total pouring time for seven tetrapods was approximately 5 1/2 hours. The balance of the 8-hour shift was used to remove and clean forms and to transport cured tetrapods to a storage area. No reinforcing was used in the tetrapods placed in the Crescent City harbor project. Utilizing reasonable care, breakage was held to a minimum and reinforcing was not necessary.

The tetrapod casting yard consisted of an 800-by 400-foot area approximately one-half of which was required for manufacturing and curing of tetrapods and the remainder for a storage area. (See Figs. 4 and 5). The casting yard was set up with a concrete batch plant located in the center of a parallel circular track system for mounting a traveling Gentry crane.

The aggregates were obtained from a commercial gravel producer on the Smith River approximately 10 miles from the casting yard. Coarse aggregates were natural river gravel. It was necessary to add blending sand and cement mill flue dust to correct a deficiency in fines in the river deposits.

Concrete mix by weight used in the manufacture of tetrapods was as follows:

Portland cement, Type II	1.00
Sand	1.61
Coarse aggregate No. 4-3/4 inches	1.77
Coarse aggregate 3/4 inch to 1 1/2 inches	1.90
Coarse aggregate 1 1/2 inches to 3 inches	2.66
Water	0.41

The water cement ratio was 4.65 gallons per sack; the air content 3.7 percent, and slump of the design mix was 3 inches. An air-entraining agent was used in the manufacture of the concrete.

Casting and Curing Procedure

Concrete was placed in the tetrapod forms in lifts approximately 20 inches deep and each lift was thoroughly vibrated before placing the next one. To prevent segregation concrete was conveyed to the lower parts of the forms by a rubber "elephant trunk". Vibration was accomplished with a 6-inch pneumatic vibrator manually operated from inside the forms. Care was taken to insure that the concrete was well vibrated down into the outer ends and along the top surfaces of the lower legs, as these points were found to be most common areas where honeycombing and rock pockets occurred.

Without using an accelerator in the concrete the top forms were permitted to be removed in 18 hours and the tetrapods lifted out of the bottom forms in five days. With 7 top and 35 bottom forms available, 35 tetrapods were cast in a 5-day work week. At the start of the job the tetrapods were cured with water for a period of 14 days; however, because of the local shortage of water and the difficulty of keeping the undersides and ends of the lower legs continuously wet, curing with a membrane curing compound was later permitted. This compound was applied as soon as the forms were removed and care was taken to assure that the membrane was not broken during the curing period. Because of difficulties encountered at the beginning of the job the contractor fell behind schedule. To correct this situation, the contractor was permitted to use a one percent calcium chloride accelerator in the concrete, thus permitting the top forms to be removed in 8 hours and the bottom forms in 60 hours. The curing period for tetrapods cast with concrete containing the

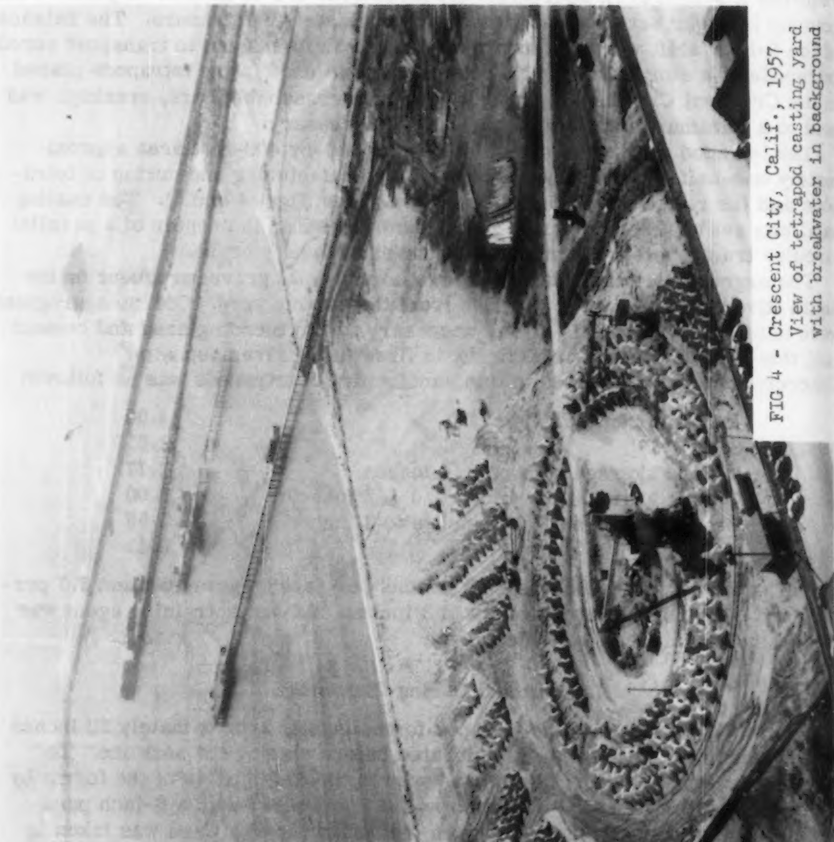


FIG 4 - Crescent City, Calif., 1957
View of tetrapod casting yard
with breakwater in background

FIG 5 - Crescent City, Calif., 1957
View taken in casting yard

accelerator was reduced to 7 days. Twenty-eight-day tests of the compressive strength of concrete manufactured with the accelerator averaged approximately 4,000 pounds per square inch.

Breakwater Construction

In constructing the breakwater, the D stone or quarry run was dumped from trucks into a skip and then placed by a Manitowac #4500 crane. After the D stone was built up to the required height for a lineal distance of about 50 feet, a layer of C stone was placed. The C stone was also dumped into a skip and placed by crane. Next in order, B_1 stone was placed on the harbor slope and crest section and B_2 stone on the crest section and seaward slope. Subsequently the A stone was built up to sea level on the harbor slope. Above that point it was necessary to place A stone and the adjacent B_1 stone simultaneously.

A tetrapod armor facing requires a solid backing to keep from ravelling. At Crescent City this was provided by a 22-foot wide, 10-foot deep concrete grouted cap. (See Fig. 6). To prevent concrete from entering the porous layer underneath the cap, the top of the B_2 stone under layer was sealed by chinking with a selected grade of D stone. The vertical face of the cap on the seaward side was provided by a heavy timber form. After placing B_1 stone between the timber form and the A stone, the cap was poured, using a transit-mix concrete grout which was well vibrated into the B_1 stone so as to provide the required 10-foot thickness of cap. Concrete in the cap contained 2 percent calcium chloride as an accelerator. The contractor poured the cap in two sections each approximately 25 feet long. The placing crane was able to move out on the freshly poured cap within three days and construct the base for the next section of the cap.

The last items to be placed were the tetrapods. The three outermost lower rows were placed first, then the two outermost top rows, and thereafter alternating lower and top rows. In order to prevent ravelling during placing the tetrapods were placed so that the last one in each row was set back from the end tetrapod in the preceding row. Following the advice of the French engineers, more tetrapods were placed in the upper layer than in the lower layer; an approximate distribution was 40 percent in the lower layer and 60 percent in the upper layer. Two advantages were anticipated from this method of placement: (1) in the event the lower layer did not fully cover the rock area, a dense upper layer was sure to fill any gaps and protect all of the underlying B_2 stone; and (2) interlocking between the two layers was improved as the dense upper layer tended to wedge into the spaces between units of the lower layer and form a tightly consolidated mass.

The experience gained on this project indicates that as long as care is taken to place the tetrapods evenly so as to obtain the required double layer and to avoid any gaps in the cover, it is not necessary to spend too much time in positioning individual tetrapods, as their shape tends to cause them to automatically interlock. Furthermore, it was noted that frequently the placing of higher rows of tetrapods caused a consolidation in the lower rows and that the first storms completed this consolidation.

A double layer of tetrapods placed on a 1 to 1-1/3 side slope to a height of about 54 feet required about 2.6 tetrapods per lineal foot of breakwater. In an 8-hour shift, approximately 70 tetrapods could be placed. Once operations were stabilized, it was possible to construct approximately 25 feet of breakwater, including cap and tetrapods, in 10 working days.

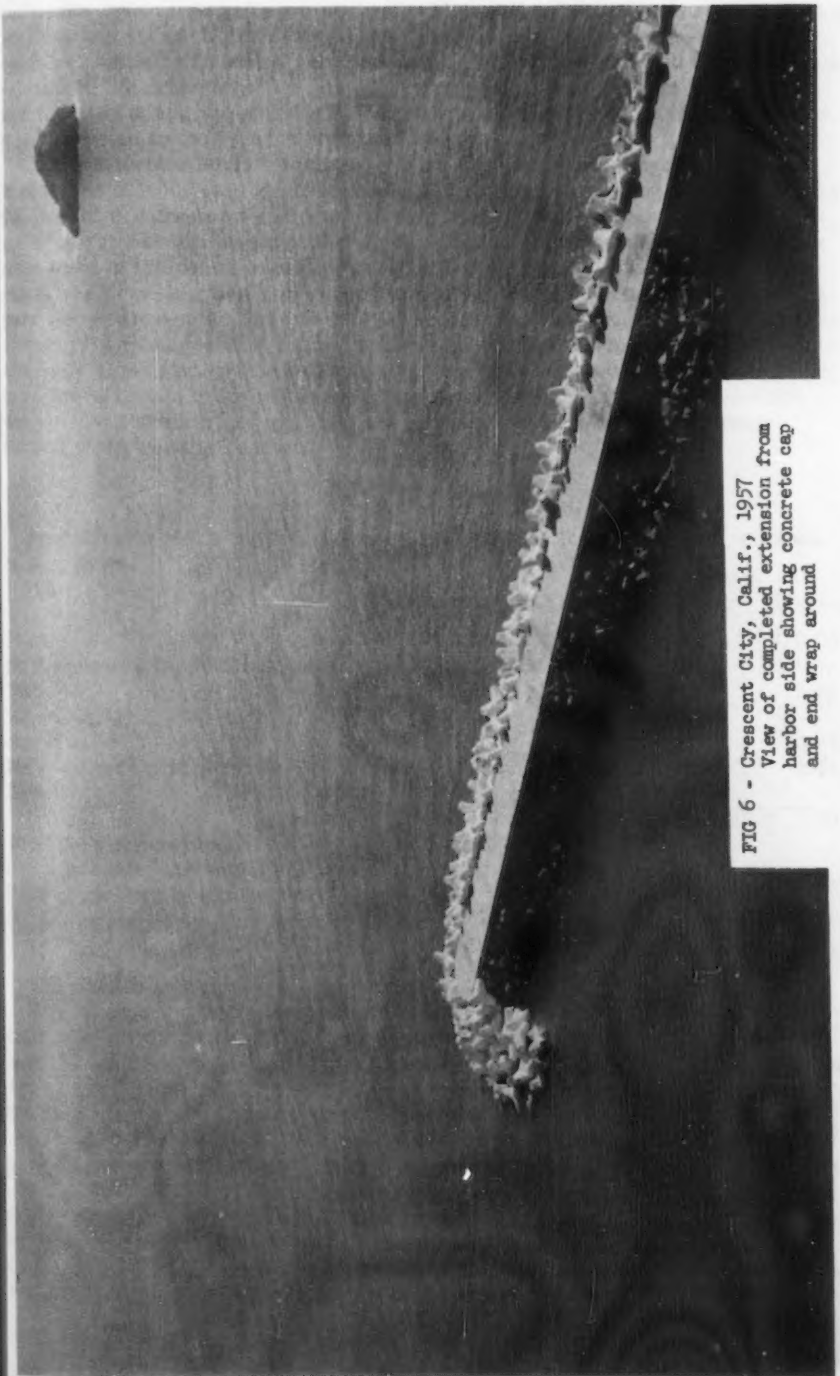


FIG 6 - Crescent City, Calif., 1957
View of completed extension from
harbor side showing concrete cap
and end wrap around

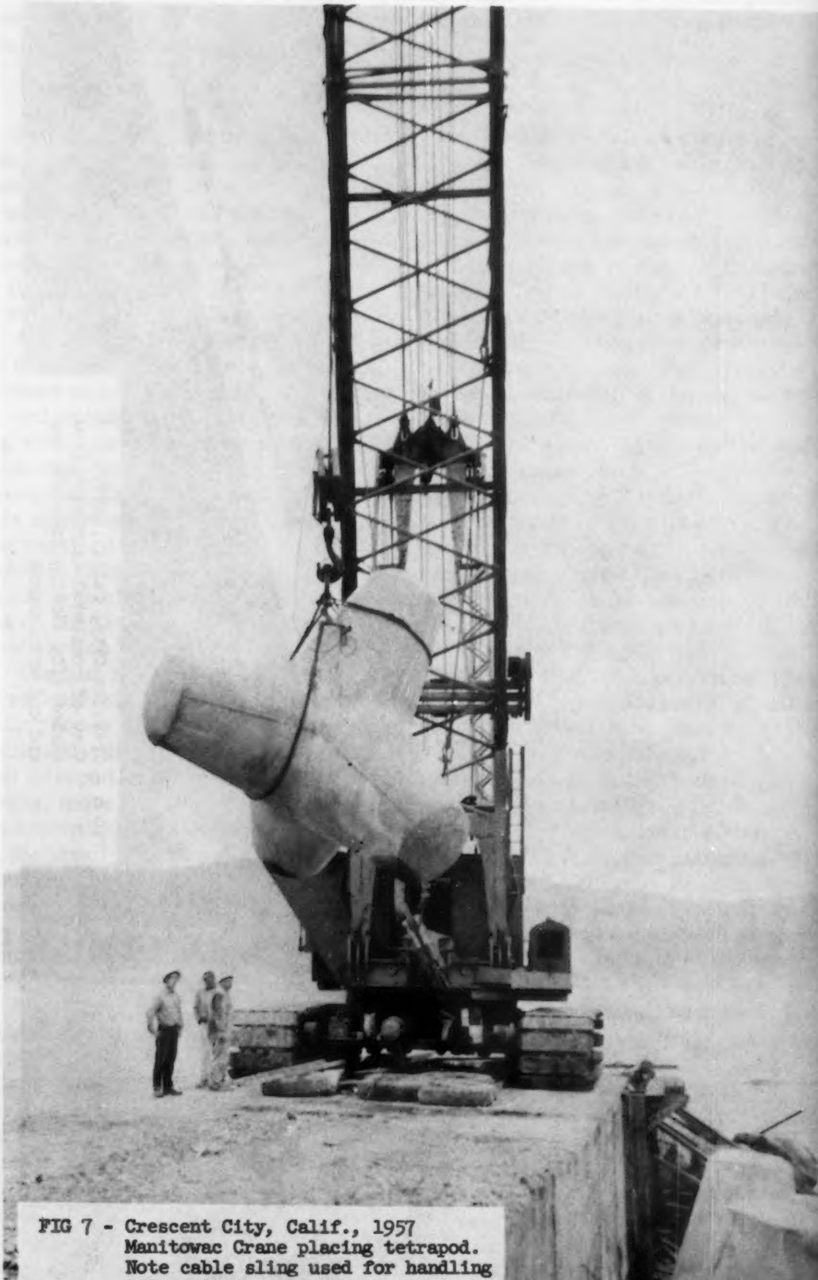


FIG 7 - Crescent City, Calif., 1957
Manitowac Crane placing tetrapod.
Note cable sling used for handling
tetrapod

The rigging used for placing the tetrapods consisted of a cable sling under one of the lower legs and a connecting cable running horizontally around the top vertical leg. (Fig. 7). One eye of the lower cable sling was connected to a quick releasing hook, which was operated by a line that extended from the hook release to the crane load line. As both ends of this load line were connected to a hoisting drum, the release was accomplished by taking up on the line that was connected to the release hook and letting out on the other line.

The breakwater section included in this contract had a length of 560 feet. It required 121,000 tons of stone, and 1,836 tetrapods. During the first season 250 feet of breakwater were constructed. A wrap-around was constructed of tetrapods at the conclusion of this first season. Approximately 315 tetrapods were used in this wrap-around with the intent that many of these could be salvaged when construction was resumed in the spring. Approximately 70 tetrapods were salvaged and re-used in the breakwater construction. In future construction of this type it is recommended that consideration be given to the use of armor stone rather than tetrapods for the interim wrap-around section. It is believed that in many instances stone would prove less expensive for the wrap-around section and might be easier to consolidate in the breakwater section when work is resumed.

Estimate of Cost

An estimate of the cost of manufacturing and placing each 25-ton tetrapod is as follows:

	Casting	Yarding	Loading & Hauling	Placing
Plant	$ 47.60	$ 3.57	$ 2.71	$ 3.00
Mobilization & Demobilization	27.56	1.10	1.01	1.00
Labor	31.48	2.14	2.79	2.59
Materials (1)	38.16	-	-	-
Supplies (2)	18.50	0.63	0.42	0.25
Distribution Costs (3)	11.81	0.45	0.42	0.44
Cement	78.02	-	-	-
	$253.13	$ 7.89	$ 7.35	$ 7.28
Total cost per tetrapod	$275.65			

(1) Aggregate, air entraining agent, etc., but excluding cement
(2) Forms, cable, etc.
(3) Superintendence, job office cost, shop, transportation, etc.

Results to Date

The storms in the Crescent City area during the winter of 1956-57 were relatively mild; no wave effect in the breakwater was noticeable. Storms of larger magnitude occurred during the winter of 1957-58 with a series of waves during April 1958 that approached the design wave height. It is believed that this last season's storms gave a fair test for tetrapod construction. In general, very little change in the breakwater was noted as a result of the high waves; however, at the very end of the structure a small amount of settlement and some movement of the tetrapods occurred. This end section at Crescent City is exposed to direct wave action from the open sea. (Fig. 8). The waves approach it from a tangential direction that tends to move the tetrapods away from the main structure; consequently when major storms occur, some movement of tetrapods may occur on this end section.

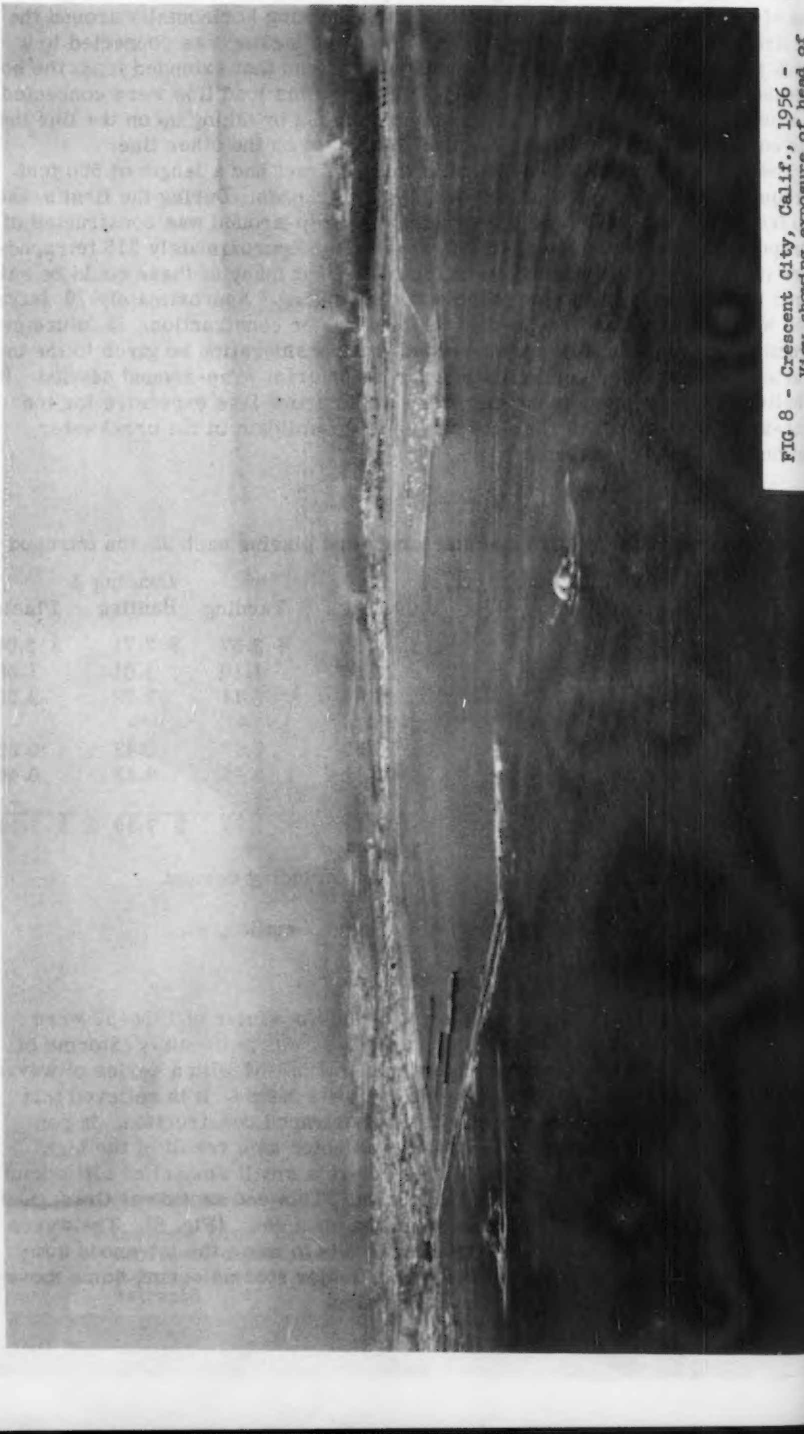


FIG 8 - Crescent City, Calif., 1956 - View showing exposure of head of breakwater to wave action

Observations and Conclusions

Tetrapods have proved a satisfactory substitute for armor stone in the construction of the Crescent City breakwater. Use of tetrapods resulted from a lack of adequate size stone within economic hauling distance. Future construction and repair of breakwaters along the west coast of the United States should consider the costs of using tetrapods or other fabricated armor facings in relation to available quarried rock. Major factors that affect these costs are as follows:

a. size of armor stone required to withstand design wave.
b. location of quarry site in relation to breakwater.
c. location and availability of casting and curing yard in relation to breakwater.
d. method of placement used - barge or truck.

It is noted that breakwater construction at Crescent City required relatively large quarried stone (A, B, and B_1 stone) in addition to tetrapods. By modification of the design, construction could have been accomplished substituting tetrapods for this stone. However, since adequate quantities of the proper size stone were available from existing nearby quarries it proved less costly to limit the use of tetrapods to the seaward armor face only. In some areas it will probably be economical to use less quarried stone and more fabricated elements, and it is conceivable that on all tetrapod or fabricated stone breakwater could be the best solution for certain areas where rock is non-existent.

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Journal of the

WATERWAYS AND HARBORS DIVISION

Proceedings of the American Society of Civil Engineers

A BREASTING DOLPHIN FOR BERTHING SUPERTANKERS

John M. Weis,[1] A.M. ASCE and Virgil Blancato,[2] F. ASCE

ABSTRACT

The design of a flexible dolphin type structure for bertning 100,000 ton supertankers in deep-water locations is presented. The application of a gravity type fender system is described and energy absorption qualities of both dolphin and fender system are discussed. Design criteria and sample energy calculations are included.

SYNOPSIS

The authors have presented a design for a dolphin type breasting structure using high tensile steel cylinders, and capable of resisting the berthing forces of 100,000 ton supertankers in semiexposed locations. The application of a gravity type fender system is described and the additional energy absorption qualities resulting from its use are discussed. Design criteria and sample energy calculations are included as a guide to be used in adapting the principle of the breasting dolphin to meet various site conditions.

INTRODUCTION

With the advent of supertankers having displacements in excess of 100,000 tons and drawing more than 40 feet of water, the port and harbor engineer is faced with the problem of providing improved structures for berthing facilities. Few existing piers have a depth of water alongside adequate for ships of

Note: Discussion open until February 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2175 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. WW 3, September, 1959.

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40 foot draft. Furthermore, few piers have been constructed to support the dredging of the bottom to 40 feet below mean low water without jeopardizing the stability of the pier. Finally, the inland location of existing facilities will require in most cases the costly dredging of deep draft channels to provide access to the ports and harbors.

It is generally accepted by most authorities in the area of waterfront structures that the best solution to this supertanker berthing problem is an island type structure, constructed in off-shore, deep water locations and fitted out with transfer facilities and pipe line connections to storage fields. In addition to providing mooring facilities, each such island type installation must possess energy absorbing qualities—not only to absorb the berthing and mooring forces, but also to absorb the energies of wave and weather action on moored vessels.

A dolphin type structure consisting of four steel cylinders and an energy absorbing gravity fender system is presented as an economical breasting facility. It is considered that varied combinations of breasting dolphins with separate piers or floating structures carrying the required transfer and pumping equipment may be adapted to meet site conditions.

Design Criteria

Design Criteria have been assumed such that the structure will be capable of absorbing normal berthing forces of a supertanker class vessel, as well as energies of excessive weather conditions attendant to berthing in an exposed area. The possibility of further growth of ships' characteristics has also been provided for in the following design criteria:

Water Depth	50 feet at mean low water
Berthing Velocity of Ship	Two knots
Displacement of Ship	135,000 gross tons
Angle of Approach with Face of Pier	10 degrees
Tidal Range	16 feet
Fender Pressure on Ships' Hull	6000 lbs per lineal foot

Energy Calculations

It is considered that the speed of two knots at an angle of 10 degrees with the face of the pier is a reasonable approach condition, considering wind, tide, current, and other possible weather conditions. Such an approach would produce a resultant velocity normal to the berth equal to 0.588 feet per second. The energy equation for a ship berthing is considered to be:

$$E_A = \frac{MV^2}{2g} \times C \qquad (1.01)$$

where: E_A = The energy of impact of the ship with the berth, in inch tons

M = The total displacement of the ship, in gross tons

V = The velocity of ship perpendicular to the berth, in feet per second

g = Acceleration of gravity

C = Coefficient of energy absorbed by pier

The coefficient C accounts for the energy lost as a result of the draft of the vessel, the hull shape, the elastic deformation in the vessel, the characteristics of the pier construction, and similar factors. For purposes of this design, this coefficient C is assumed to be 65 per cent. That is, it is assumed that 35 per cent of the potential energy of impact is dissipated, leaving 65 per cent of the total berthing forces to be absorbed by the pier.

Simple substitution of design conditions in Eq. (1.01) gives;

$$E_A = \frac{135,000 \times .588^2 \times 12 \times .65}{64.4} = 5700 \text{ inch gross tons} = 6350 \text{ inch tons}$$

Description of Breasting Dolphin

The basic structural element is a 48 inch diameter high-tensile steel cylinder, having a maximum wall thickness of 1-1/2 inches. The breasting dolphin consists of four such cylinders, spaced 40 feet on centers and inter-connected by a frame, brace and waler system as shown in plan on Plate 1. Dimensions of the frame members are variable to provide a platform of the desired width. Brackets are attached to the walers at 10 feet intervals to support a fender system along each side of the dolphin, and eight sets of brackets are installed on each of the curved ends to support the end fenders.

This structure provides an energy absorbing berthing unit 120 feet in length. Two or more of these units, as necessary to accommodate the ships utilizing the facility, may be combined to protect the piers or floating structures supporting the pumping and transfer equipment. Bollards are included as an integral part of each cylinder to provide the necessary anchorage for mooring lines. The necessity of anchor buoys and their location for bow and stern lines is considered optional as site conditions may dictate. These conventional anchor buoys are not included in the design as a component of the breasting dolphin structure.

Dolphin Cylinder Design

The moment of inertia of a 48 inch diameter cylinder with a wall thickness of 1-1/2 inches is 59,300 inches,[4] and the section modulus is equal to 2,470 inches.[3]

For design purposes, it was assumed that the 120 feet long cylinders were driven in water having a depth of 50 feet, and the point of fixity was assumed to occur 20 feet below the mud line. It is recognized that this point of fixity may vary according to the characteristics of the soil. It was further assumed that the point of application of the berthing forces is 10 feet above mean low water, giving an effective cantilevered beam length of 80 feet. As a result of the load distributing characteristics of the brace and waler system, it may be assumed that berthing forces are distributed to all four cylinders. Thus the center cylinders may be assumed to carry between 25 and 30 per cent of the berthing load and each of the end cylinders are assumed to carry the remaining 20 to 25 per cent.

Considering each cylinder as a cantilevered beam having a yield point of 50,000 psi, a section modulus of 2,470 cubic inches, and the load applied at a distance of 10 feet above mean low water, the expression for the load P is:

$$P = \frac{S \times f_s}{L} \qquad (1.02)$$

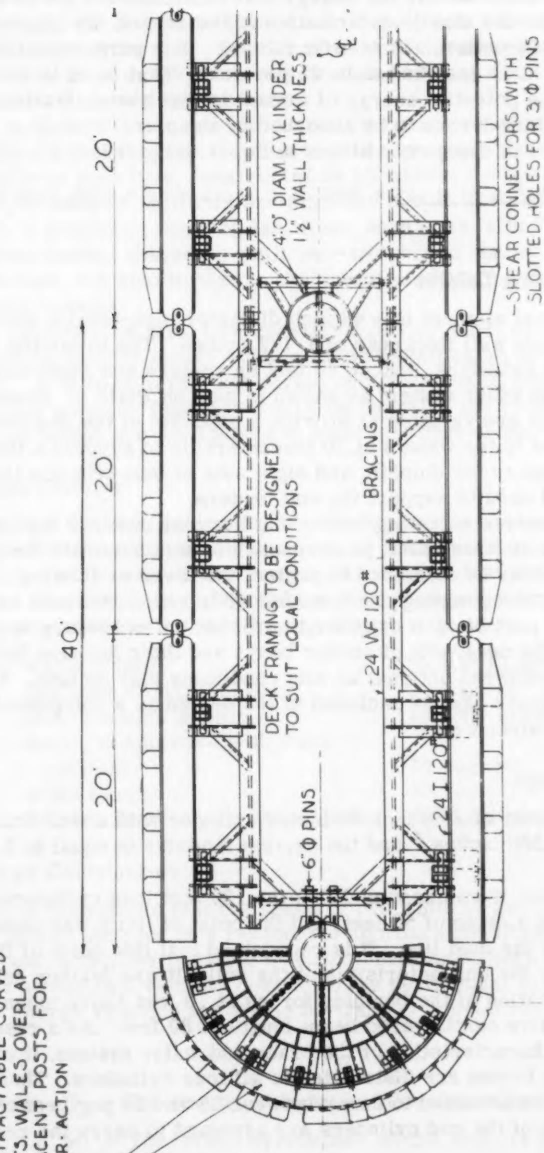

BREASTING DOLPHIN PLAN
4 RETRACTABLE CURVED UNITS, WALES OVERLAP ADJACENT POSTS FOR INTER-ACTION
6" ϕ PINS
24 I 120
24 WF 120#
DECK FRAMING TO BE DESIGNED TO SUIT LOCAL CONDITIONS
BRACING
4'0" DIAM CYLINDER 1½" WALL THICKNESS
SHEAR CONNECTORS WITH SLOTTED HOLES FOR 3" ϕ PINS
20'
40'
20'
20'

PLATE 1

and the maximum value is computed to be:

$$P = \frac{2470 \times 50{,}000}{80 \times 12} = 129 \text{ K, or } 65 \text{ tons}$$

The expression for the deflection of this cantilevered beam is:

$$\delta = \frac{PL^3}{3EI} \tag{1.03}$$

Substituting in Eq. (1.03),

$$\delta = \frac{129{,}000 \times 80^3 \times 12^3}{3 \times (30\times10^6) \times 59300} = 22 \text{ inches}$$

Thus the total energy absorption capacity of each cylinder is equal to:

$$E_R = \tfrac{1}{2}\, P\delta \tag{1.04}$$

$$E_R = \tfrac{1}{2} \times 65 \times 22 = 715 \text{ inch tons}$$

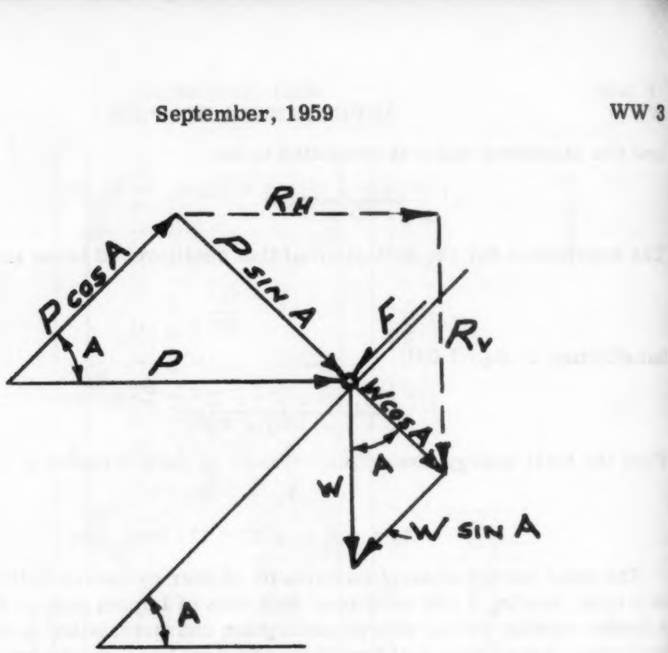
The total energy absorption capacity of four cylinders will then be 2,860 inch tons, leaving 3,490 additional inch tons of impact energy to be absorbed. A fender system having energy absorption characteristics is considered the most economical means of providing additional energy absorption in the structure. After considering the various gravity and resilient type fenders, an adaptation of the retractable fender system as developed at the New York Naval Shipyard(1) was selected. Not only does this fender provide protection for berthing ships, but also provides the additional energy absorption qualities required by the structure.

Retractable Fender Principle

The retractable fender system is based on a gravity principle but provides additional energy absorption qualities as a result of friction between the fender and the brackets. Recognizing the need for a fender system capable of retracting as much as twenty inches, certain modifications were made to the original retractable fender system design.

The first major modification involved the linking of sections of the fender system, instead of providing a continuous fender system. This linking provides a chain-like system in which resistance to movement increases proportionately with the number of sections retracted, and therefore, in direct proportion to the acting force. The second major modification involved a redesign of the bracket system. Instead of using fabricated plate brackets attached to the pier, a system of angles welded as shown on Plate 2 was adopted. These modifications while preserving the basic principle of the retractable fender system, contribute greatly to the construction procedures, and facilitate future repairs and replacement of worn out timbers.

The amount of outside force required to move the fender inward and upward is directly proportional to the weight of the fender, the angle of inclination of the sliding surface and the coefficient of friction of the sliding surfaces. A free body diagram of the supporting pin may be considered to be:



From this free body diagram it may be seen that in order for a body of weight W to be moved inward and up the inclined plane by the horizontal force P, the component (P cos A) must be greater than (W sin A + F) where F is the frictional resisting force equal to f (P sin A + W cos A)

$$\therefore P \cos A > W \sin A + f (P \sin A + W \cos A) \tag{1.05}$$

Assuming an angle of inclination of 45 degrees and a factor of friction of steel on steel of f = .30, the horizontal force P required to move the weight W must be greater than 1.85 W. Further, assuming the factor of friction of steel on steel as f = .30, the value of P for other angles of inclination will be as follows:

A = 40 degrees; P = 1.52 W
A = 35 degrees; P = 1.25 W
A = 30 degrees; P = 1.05 W

A particular advantage derived from the use of the retractable fender system in this design is the dissipation of a large amount of kinetic energy through friction. As a result of this dissipation, the potential energy of the system in its retracted position is less than 54 per cent of the energy absorbed in the displacement of the fender. An additional 20 per cent of the potential energy is dissipated by friction in the descent of the fender. This eliminates the need for protection of the ship upon the return of the fender to its stable position.

It may be seen that a force equal to almost twice the weight of the fender is required to move the fender over a surface inclined at 45 degrees. This movement of the fender absorbs energy and contributes to the overall energy absorption capacity of the dolphin.

If each section of the fender was independent, when the acting force P equalled 1.85 W the fender would move under a constant force. However, if the sections are linked together the adjacent sections will impose increasing resistance to the movement, requiring a proportionate increase in the acting force to obtain the movement. This condition will continue until the various sections come in contact with the vessel, thus applying a direct force to each section.

For purposes of determining the amount of energy absorbed by the fender, it may be considered that each unit is moved individually and uniformly from the beginning to the end of the travel. Thus, the total energy absorbed by the fender may be considered to be:

$$E_{RF} = 1.85\ W \times d \tag{1.06}$$

where W is the total weight of the fender and d the distance of travel or retraction of the fender.

Description of the Fender

Substituting in Eq. (1.06) it becomes evident that in order to provide a fender capable of supplying 3,490 inch tons of energy absorption, it will be necessary to provide a fender of approximately 100 tons retracting a distance of 20 inches. Six sections, each 20 feet long and weighing over 15 tons each, will provide the necessary weight and are adaptable to the dolphin structure.

As shown in Plate 2, each unit is a rigid frame composed of three walers of 24 inch, 120 pound I sections connected to two 12 inch, 106 pound wide flange vertical posts. Each post is provided with two 4 inch diameter bearing pins. Hard wood contact fenders spaced four feet apart, with chocks between, are bolted on the outboard flange of the wales. These timbers provide a continuous contact surface between the tanker and the dolphin. To increase the weight of the fender, precast concrete blocks of about 8 tons total weight for each unit are placed over the wales and fastened to the posts.

Each fender unit is connected to the adjacent unit by three shear connectors with slotted holes and 3 inch diameter bolts. The single bolt in a slotted hole gives sufficient freedom to each unit for a specified amount of independent movement before being restrained by adjacent units.

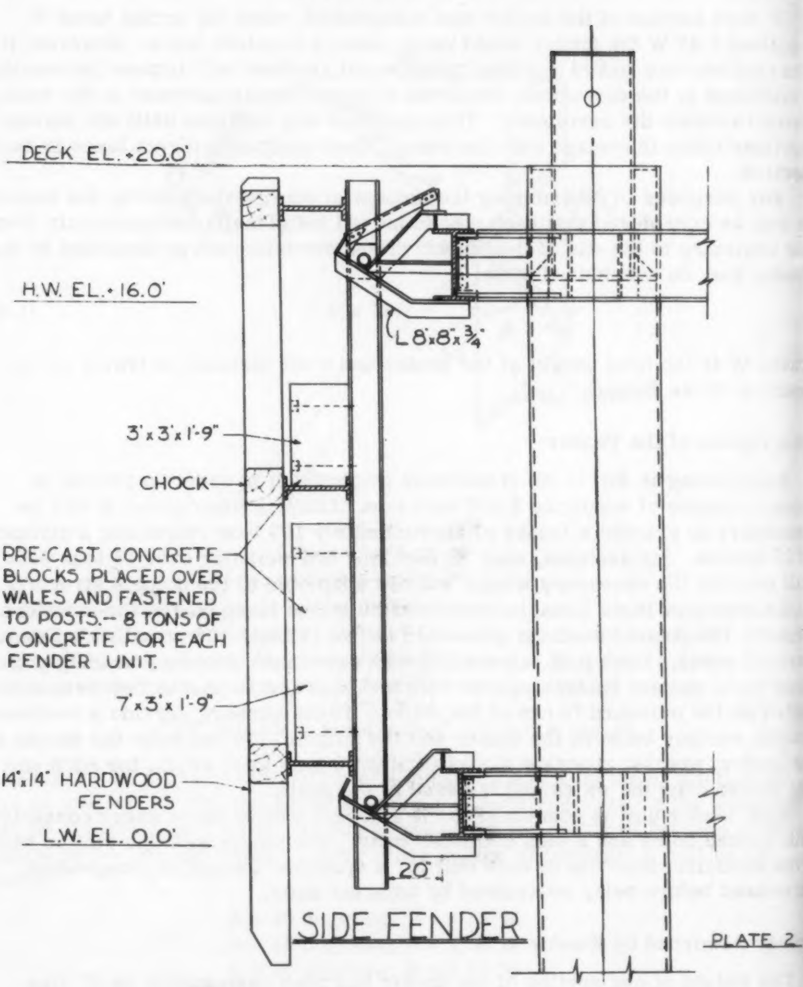
Energy Absorbed by Fender

The weight of one section of the fender has been computed to be 37 kips. Thus, the weight requirement set forth in the preceding paragraphs has been fulfilled.

The force P required to move one 20 foot section up on the bracket equals 1.85 W or 68.5 kips. By substitution in Eq. (1.06), the energy absorbed by this one unit moving a distance of 20 inches is computed to be:

$$E_{RF} = 68.5 \times 20 = 1{,}370 \text{ inch kips or } 685 \text{ inch tons}$$

The six units will give a total energy absorption of 4,110 inch tons. These calculations do not take into consideration the internal energies that may be absorbed by the deflection of the various wales, fixed and movable, and the contacting timbers. It also does not take into consideration the effect of buoyancy on the weight of the movable fender or the resistance of the fender to movement through the water at various tide conditions. These effects tend to



SIDE FENDER

PLATE 2

counteract each other and are not considered to be of sufficient significance to be further considered.

Reaction of Dolphin

Considering the free body diagram of the supporting pin (Fig. 1) it may be seen that the only forces transmitted to the cylinders by the bracket system are those components of the berthing force P and the weight of the fender W acting normal to the inclined plane of the bracket; i.e. (P sin A and W cos A).

Thus, a force (P sin A + W cos A) is transmitted to the cylinders at an angle equal to the angle of inclination of the sliding surface of the bracket. The

horizontal component R_h and a vertical component R_v of this force may be expressed as:

$$R_h = (P \sin A + W \cos A) \sin A \quad (1.07)$$

$$R_v = (P \sin A + W \cos A) \cos A \quad (1.08)$$

Substituting values of 1.85 W for P, and considering A equal to 45 degrees it may be seen that

$$R_h = R_v = 1.42\ W$$

These two forces cause opposite moments on the cylinders of the dolphin. The moment caused by the horizontal force cantilevered at a distance of 80 feet above the point of fixity acts in a clockwise direction, whereas the moment caused by the vertical component of the force, applied in this case 7 feet away from the center line of the cylinders is counterclockwise. Considering the six fender units along one side of the breasting dolphin to be in equilibrium under the acting force P, the clockwise moment M_h may be computed to be:

$$M_h = 1.42\ W \times 80' \times 6 + 25200 \text{ foot kips}$$

and the counterclockwise moment M_v is found to be:

$$M_v = 1.42\ W \times 7' \times 6 * 2200 \text{ foot kips}$$

Therefore, the resultant moment on the dolphin cylinders is 23,000 foot kips at the point of fixity. The horizontal force necessary to cause this moment of 23,000 foot kips in the dolphin cylinders would be equal to $\frac{23000}{80} = 288$ kips = 144 tons. Thus, it is evident that the effective horizontal force transmitted to the structure by an acting force of 205.5 tons (sufficient to retract six fender units) is only 144 tons on the cylinders. This horizontal force of 144 tons would cause a deflection of the cylinders of only 12 inches.

As noted from Eq. (1.02), the allowable load P for each cylinder is 65 tons. Thus, the total horizontal force required to stress four cylinders to the yield point is 260 tons, with a resulting deflection of approximately 22 inches. Therefore, when the fender is in its uppermost position, and the dolphin has deflected only 12 inches, an additional horizontal force of 116 tons would be required to deflect the dolphin to its maximum permissible deflection. With the fender retracted and the dolphin deflected 12 inches, the total energy absorbed by the structure may be computed to be:

$$E_1 = \frac{P\theta}{2} + Pd \quad (1.09)$$

$$\therefore E_1 = \frac{205.5(12)}{2} + 205.5(20) = 5343 \text{ inch tons}$$

In order to obtain the maximum permissible deflection of the cylinders the force of 116 tons for the additional 10 inches deflection would be required. The increased energy absorption would then amount to:

$$E_2 = \frac{(116 + 205.5)(10)}{2} = 1605 \text{ inch tons}$$

The total energy absorption capacity of the structure is therefore:

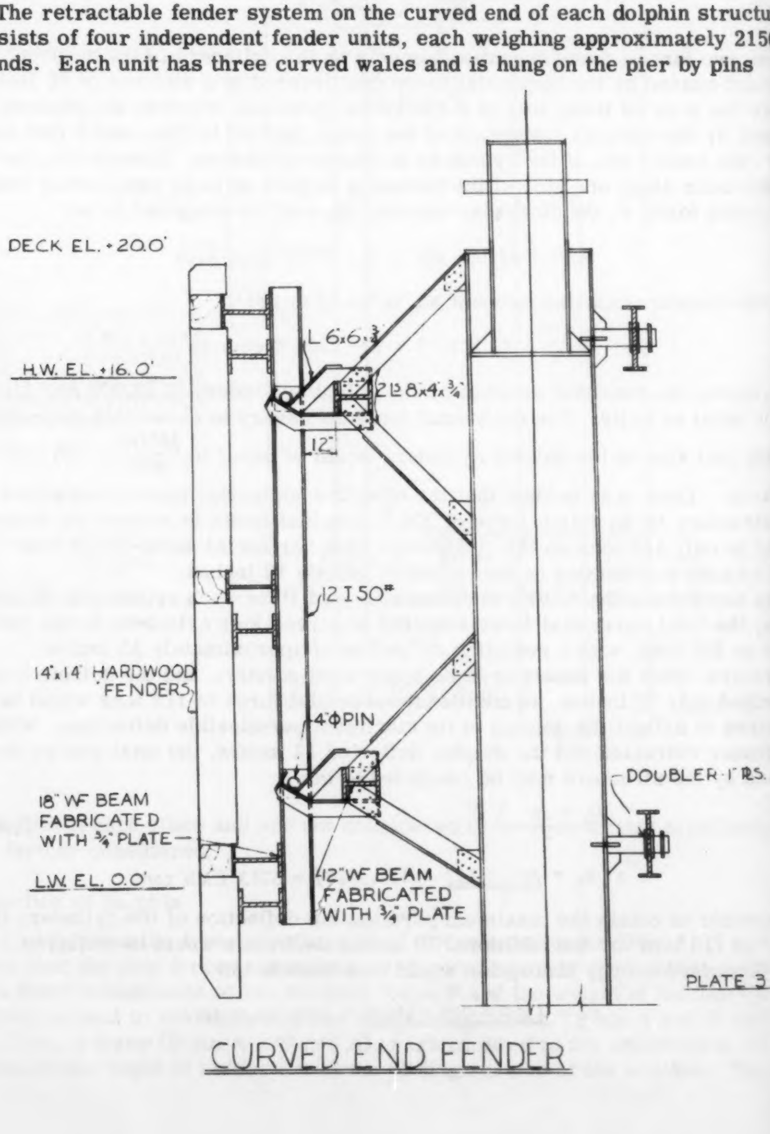
$$E = E_1 + E_2 \tag{1.10}$$

$$\therefore E = 5343 + 1605 = 6948 \text{ inch tons}$$

A similar breasting dolphin provided with a rigid fendering would have a maximum energy absorption capacity of only 2860 inch tons.

Curved End Fender

The retractable fender system on the curved end of each dolphin structure consists of four independent fender units, each weighing approximately 21500 pounds. Each unit has three curved wales and is hung on the pier by pins



CURVED END FENDER

bearing on brackets as shown on Plates 1 and 3. Timber sheeting is fastened on the outer flange of each wale.

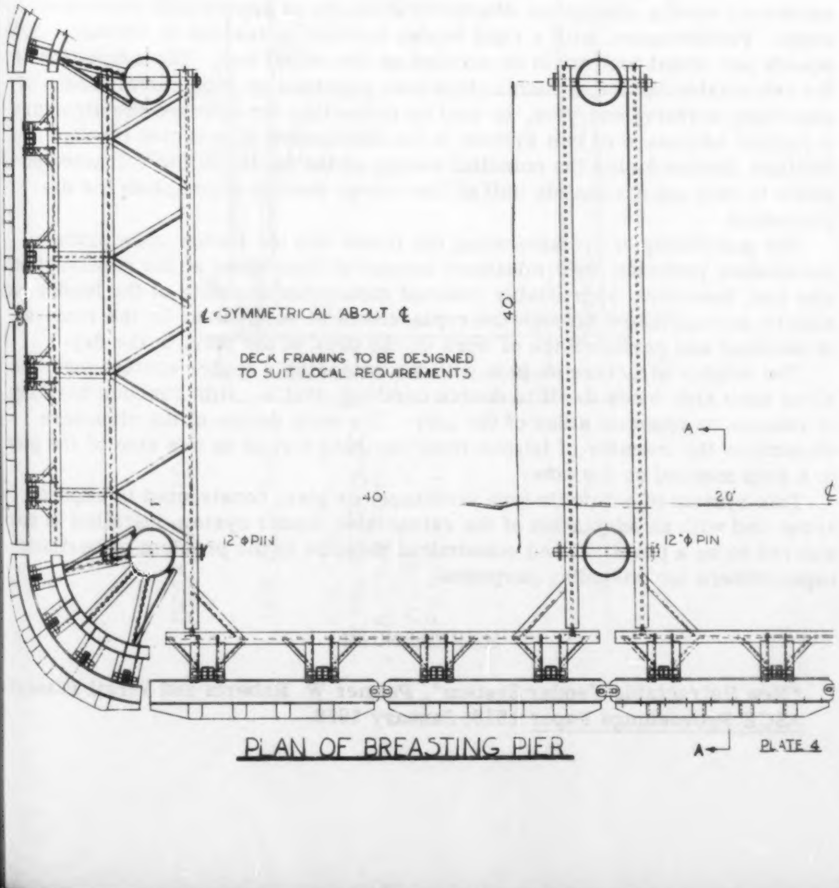
Each frame is limited to a 45 degree arc, to facilitate retraction in a radial direction. The wales of each frame extend greater than 45 degrees to overlap the posts of the adjacent frames. When one frame is under pressure and starts to retract, the overlapping wales force the adjacent frame to follow the movement in the same direction. With this arrangement, at least two curved units will move together, increasing considerably the resistance to the acting force. This also avoids the interlocking effect of the frames as would otherwise be caused by radial movement toward the center.

It is assumed that two of the four independent units will retract for a distance of 12 inches when struck by a ship. The horizontal force necessary to cause this retraction is equal to 1.85 (43) = 79.5 kips.

Thus, the fender and the end cylinder are capable of absorbing approximately 2380 inch kips from a ship striking the end of the dolphin structure.

Application of Flexible Cylinders in Off-Shore Breasting Piers

By using two lines of cylinders of appropriate section modulus, a pier of about 60 feet width, as shown on Plate 4, has been designed to support



PLAN OF BREASTING PIER

PLATE 4

self-contained pumping and transfer equipment. A gradual cushioning of berthing forces is effected by the manner in which the pier framing transmits the load to the cylinders. The load is first taken up by the line of cylinders nearest to the vessel and then, when the berthing force is sufficiently large, by the second line of cylinders at the opposite side of the pier. In this application, the rigid frame of the pier deck is pin connected to the cylinders with slotted holes 10 inches long (see Plate 5). With this type of connection the cylinders at the ship side will deflect for a distance of about 10 inches before the other line of cylinders will start to deflect; thus increasing the resistance to the acting force and at the same time providing a larger safety factor for the entire pier.

SUMMARY

As evidenced from the preceding discussion a dolphin type breasting structure, or pier, using high tensile steel cylinders and a retractable fender system is a solution to the supertanker berthing problem in offshore locations.

The retractable fender system, as designed, is capable of absorbing approximately 75 per cent of the berthing energies for a supertanker of 135,000 gross tons. The use of a rigid fender in this same design would require cylinders of an uneconomical diameter and increased section moduli providing the necessary energy absorption characteristics but at appreciably increased costs. Furthermore, with a rigid fender system, pressures in excess of 6000 pounds per lineal foot would be exerted on the ships' hull. The adaptation of the retractable fender system in this case provided an economical means of absorbing berthing energies, as well as protecting the ship and the structure. A further advantage of this system is the dissipation of energies through friction; thus reducing the potential energy of the fender in the retracted position to only approximately half of the energy used to accomplish the displacement.

The possibility of prefabricating the frame and the fender units prior to installation provides for a minimum amount of time spent at the construction site and, therefore, appreciably reduced costs. Maintenance of the fender may also be accomplished through the replacement of sections or by the removal of sections and performance of work on the deck of the pier, in the dry.

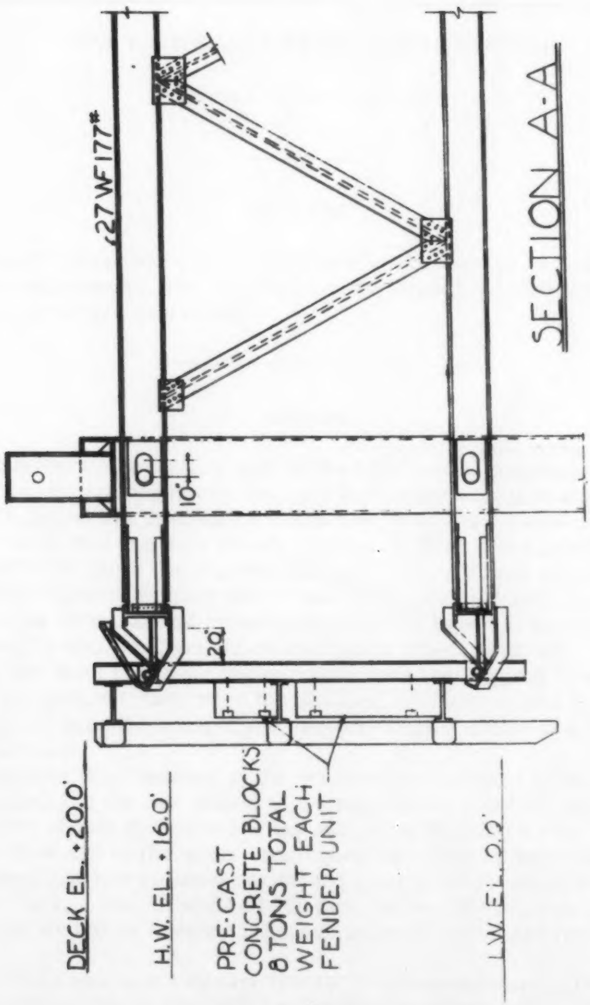
The dolphin structure or pier with the retractable fender system installed along each side lends itself to double berthing; that is, simultaneous berthing of vessels on opposite sides of the pier. The very design of the structure eliminates the transfer of impact from berthing forces on one side of the pier to a ship moored on the other.

This system of a dolphin type structure, or pier, constructed in exposed areas and with an adaptation of the retractable fender system installed is considered to be a practical and economical solution to the problem of berthing supertankers for unloading purposes.

REFERENCES

1. "New Retractable Fender System", Palmer W. Roberts and Virgil Blancato, ASCE Proceedings Paper 1513, January 1958.

PLATE 5


SECTION A-A
℄ 27 WF 177#
10'
20'
DECK EL. +20.0'
H.W. EL. +16.0'
PRE-CAST CONCRETE BLOCKS 8 TONS TOTAL WEIGHT EACH FENDER UNIT
L.W. EL. 2.2'

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Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

NEW YORK STATE BARGE CANAL SYSTEM

Edward C. Hudowalski[1]

ABSTRACT

This Canal System has contributed much to the welfare of New York State. Its history, engineering, and its effects on the economy of the State and the Nation as a whole are described.

History

Three quarters of a century ago, at the 1884 annual convention of the ASCE, a score of prominent engineers, led by Elnathan Sweet, State Engineer of New York, participated in a discussion of the New York State Canal question. By that year, the Grand Ole Erie Canal, "Clinton's Ditch," had paid for itself; it had supported the State Government for many years; it had brought affluence, prosperity and growth to the State; it had developed business, commerce and industry in the terminal and intervening cities so they had developed and grown. Yet, at that time, traffic on the canals was languishing. Even though the canals had been improved and enlarged over the original Erie, they were becoming obsolescent; they were too shallow, too narrow, and had locks that were too small for the trends in the then-developing steam-engine-propelled barges and boats.

Small wonder that, because of the benefits that accrued to the State from the Erie Canal and the new trends in transportation, the 1884 annual meeting of the Society should be devoted to the subject of building a ship canal across New York State and to the whole broad question of inland waterways. The canal project that was outlined envisioned a canal which would have a depth of 18 feet of water, a bottom width of 100 feet, locks 450 feet long by 60 feet wide, and be so located as to have a continuous descent from Lake Erie to the

Note: Discussion open until February 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2176 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. WW 3, September, 1959.

1. Asst. Supt. of Operation and Maintenance, New York State Dept. of Public Works, Div. of Operation and Maintenance, Canals, Waterways and Flood Control, Albany, N. Y.

Hudson River. The thinking was directed towards a deep waterway, lying entirely in the United States, which would connect the Great Lakes with the Atlantic Ocean.

That discussion effected no result, except that it did help to mould public opinion favorable to inland waterways and was significant in portraying the trend of thought, which, many years later, became crystalized in actual accomplishment—the New York State Barge Canal.

Early Historical Background

In the early sixteen hundreds, the Dutch and the English established settlements on the Hudson and Mohawk Rivers. The French did likewise on the St. Lawrence River and the Great Lakes. Through the early development years of our continent and country, the streams and rivers between these great waterways and places became the routes of trade, commerce and war. The early traders and voyageurs proved this area in New York to have the most favorable conditions for joining by water the Great Lakes and the interior of the country with the Atlantic Ocean that could be found any place along the Atlantic seaboard. Here the westerly range of the Alleghany Mountains disappears as it approaches the Mohawk Valley, and the easterly range is cut by the Hudson River. George Washington noted the potential wealth of the surrounding country and the feasibility of inland navigation, while traversing this region in 1783 on his grand tour of the State, and remarked that this was the seat of empire. The "Grand Old Erie" and the enterprise of the early New York Staters made that prophecy of George Washington come true, for New York State is today the Empire State in name and in fact.

The early water transportation and inland lock-navigation companies made use of this trade route. The Mohawk was followed to "the carry" near Fort Stanwix (Rome). After the portage, Wood Creek was followed to Oneida Lake, then the Oneida and Oswego Rivers to Oswego and Lake Ontario. Deeper penetration into the State by these transport companies was made by way of the Seneca and Clyde Rivers and Canandaigua Creek. These early transportation ventures depended on the natural streams and lakes. They failed because the sponsors were unsuccessful in taming the rapids, falls and periodic low water in the streams. They succeeded, however, in focusing attention on the feasibility of developing a water route to the interior of the country.

The early inland lock-navigation companies made use of the natural water courses except at rapids and waterfalls. Such blocks to navigation were first bypassed by portage, and sluiceway and later by locks after they were invented in Europe. The planners of the Erie Canal decided on the so-called English rule of digging an artificial navigation channel independent of the natural streams and lakes.

The Erie Canal, The First School of Engineering in America

When the building of the Erie Canal was begun in 1817, there were no contractors as such. The profession of engineering was practically unknown in America. Excavating and earth moving machines were yet to be invented. The only construction equipment available was the pick, lever, spade, wheelbarrow and wagon. The ingenuity of the early canal diggers was demonstrated shortly after work began by the manner in which the contractors and work superintendents improvised tools and methods to speed up construction. The plow and scraper helped to speed up excavation. Cement mortar was discovered to

make the locks and other structures stronger and leak-proof. The block and fall and a wheeled windlass speeded up downing of trees and removal of stumps. Someone discovered that the movements of horses and wagons on the towpath and berm did a much better job of compacting and stabilizing them than tamping.

The foreign engineers, who were asked by the Canal Commissioners to take on the job of building the Erie Canal refused, even after fantastic salaries were offered. This resulted in the employment of eminent New York surveyors and others of its citizenry to plan and progress the work of building the canals. The State, therefore, calling on her own citizens, developed, not only canal commissioners, but contractors and engineers, who, in 1825, in the words of Canal Historian, N. E. Whitford, ". . . have built the longest canal in the world in the least time, with the least experience, for the least money, and to the greatest public benefit." Thus it can rightfully be said that the Erie Canal, again quoting Whitford, ". . . the great pioneer work of engineering in America, was the first American School of Engineering."

Engineering Significance Recognized

The success story of the Erie Canal spread like wildfire through the country and the world. A demand for canal-trained engineers was created even beyond the supply. All engineers, who had a part in the building of the Erie, were called to "engineer and build" other great works throughout the country. The younger men trained in canal maintenance and improvements and passed on to give their talents to the building of railroads. Thus, the first school of engineering—The Grand Old Erie—graduated the engineers, who lined the country with their works of internal improvements.

It has been sagely queried, said Historian Whitford, ". . . whether it was the building of the Erie which gave birth to engineering, or did the spirit of engineering, groping for expression, create the canal?"

In his presidential address before the American Society of Civil Engineering in 1899, Mr. Desmond Fitzgerald recognized the significance of the Erie Canal to engineering when he discussed the history of engineering in this country. He divided this history into four periods. These periods he portrayed as that of: Canal agitation and experimentation; canal building; railroad building and modern engineering. Mr. Fitzgerald had these periods running respectively from 1785 to 1810, 1810 to 1830, 1830 to 1848, and from 1848 to the date of his address.

Need for Improvement

Traffic on the obsolescent Erie and other New York State Canals was languishing. Railroad traffic and rates were zooming upwards. Traffic deals between railroads, providing for monopoly and rate differentials to Atlantic seaboard cities, was the vogue. Such practices resulted in increased port activities in cities like Boston, Providence, Philadelphia, etc., and decreased in New York City. The State Comptroller and New York business men were alarmed over such practices of the railroads. They recognized that competition was the only weapon with which such rate deals could be fought. Tolls were gradually reduced and finally abolished and certain improvement work on the obsolescent canals was inaugurated. These actions had their salubrious effects in the Port of New York City and on industry and business throughout

the State, for rail rates were reduced proportionately to the reduction in tolls and the amount of improvement work performed.

Public opinion was therefore developing for the enlargement and improvement of the canals. The discussion of the canal question at the 1884 annual meeting of the Society resulted in the New York Board of Trade organizing a State convention to consider permanent improvements to the State waterways. "The Union for the Improvement of the Canals of the State of New York" was organized at this convention, which was held in Utica in 1885. The "Union" gained strength as it gathered into its membership powerful and influential commercial and industrial interests within the State. The Union was instrumental in a great deal of improvement, enlargement and modernization work on the canals. It influenced the 1892 Legislature, which authorized the election of delegates to the 1894 Constitutional Convention and stipulated that one of the convention duties be ". . . consideration of amendments relating to the care of improvement of the State Canals."

The decade that followed this Constitutional Convention was charged with tremendous fervor of canal improvement and enlargement agitation activities. Practically the entire Nation got into the game of canal building in New York State. The State Engineer, U. S. Army Engineers, Congress, professional engineers, promoters—all had ideas on what the canals should be. Studies, surveys, investigations, research, reports and recommendations were cropping up by the dozens with considerations from enlarging the canals to the construction of a 30-foot-deep ship canal across the State.

Ship or Barge Canal?

Throughout the discussions and thinking on the canal question, the emphasis was being placed on a ship canal. Much of such thinking was visionary rather than practical. The Rivers and Harbors Act of June 3, 1896, directed the Corps of Engineers, U. S. Army to make " . . . accurate examination and estimate of cost of construction of a ship canal by the most practicable route wholly within the United States from the Great Lakes to the navigable waters of the Hudson River, of sufficient capacity to transport the tonnage of the lakes to the sea." This task was given to Colonel Thomas W. Symons, who went beyond the call of duty and submitted a report, which included a study of comparative costs and benefits of a ship canal versus a barge canal. His recommendation, which was ultimately adopted by the State, called for a barge canal for boats of 1,500 ton capacity. Amidst the prevailing confusion on the canal question, this recommendation appeared as a solution to the problem; was received with favor; and grew in estimation in time. It can be said that Colonel Symons' report became the basic principle, which influenced the thinking in favor of a barge canal, and ultimately governed the State engineer's choice of a barge canal across New York State.

The 1901 State Engineers' Report outlined the dimensions and routes of the barge canal system. It indicated the canalization of natural streams and lakes wherever possible. Land channels were indicated where no natural waterways existed, and at locations in streams where the alignment would call for a cut across a river bend. Considered from an engineering standpoint, this report left little to be desired. This report set up the barge canal.

Political Contention Over Canal Question

All was not smooth sailing, however, once the barge canal idea took hold. In 1901, Governor Odell placed the canal problem before the Legislature. The questions were:

1. Shall the canals be abandoned?
2. Shall the canals be enlarged to accommodate 1,000-ton barges?
3. Shall the widening and deepening improvement work, begun under the 1895 Improvement Act, continue?

There followed two years of bitter controversy. Opponents of the canals were desperately waging a campaign to discredit canals and force their abandonment. Dissensions as to the size of canals were rife in the ranks of the advocates of canals. The question of restoration of tolls on the canals crept in. For two years, it looked like the opponents of canals would win out.

During this time organizations, such as the very influential Canal Association of Greater New York—comprised of the leading commercial interests of New York City and State—and others—fought the anti-canal interests; united the canal advocates; influenced both political party conventions in 1902 to adopt a barge canal plank in their platforms; and, through untiring efforts of education and promotion, saw the barge canal referendum bill pass through the Senate and Assembly during the 1903 legislative session, and the barge canal became a fact by one quarter of a million vote majority during the 1903 November elections. The people of the State accepted the barge canal, voted one hundred and one million dollars for its construction and reaffirmed the 1883 decision that the State canals remain toll-free.

Thus the start of this century saw another great waterways project commenced in New York State. The design and building of the barge canal was another milestone in the progress, growth and development of the profession of engineering.

Barge Canal System Described

The New York State Barge Canal System comprises four divisions. (See Fig. 1)

Champlain Division

The Champlain Canal starts at Troy. It utilizes the canalized Hudson River to Fort Edward. From Fort Edward to Whitehall, this canal is an artificial land cut channel. From Whitehall to Rouses Point, the waters of Lake Champlain are used. This Division has 11 locks from Waterford to Whitehall.

The Champlain Canal follows the northern pathway of the history of the Nation. Along this route passed the early fur traders and the trade and commerce between New York and Montreal. Armies of the war between the Colonies and England trod back and forth on this pathway. Tributary streams of the Hudson River and Lake Champlain provided a most natural highway for bateau between these waterways, with easy portage.

Erie Division

The Erie Division stretches from the Hudson River at Troy across the State to Tonawanda on the Niagara River. This canal uses the Mohawk River

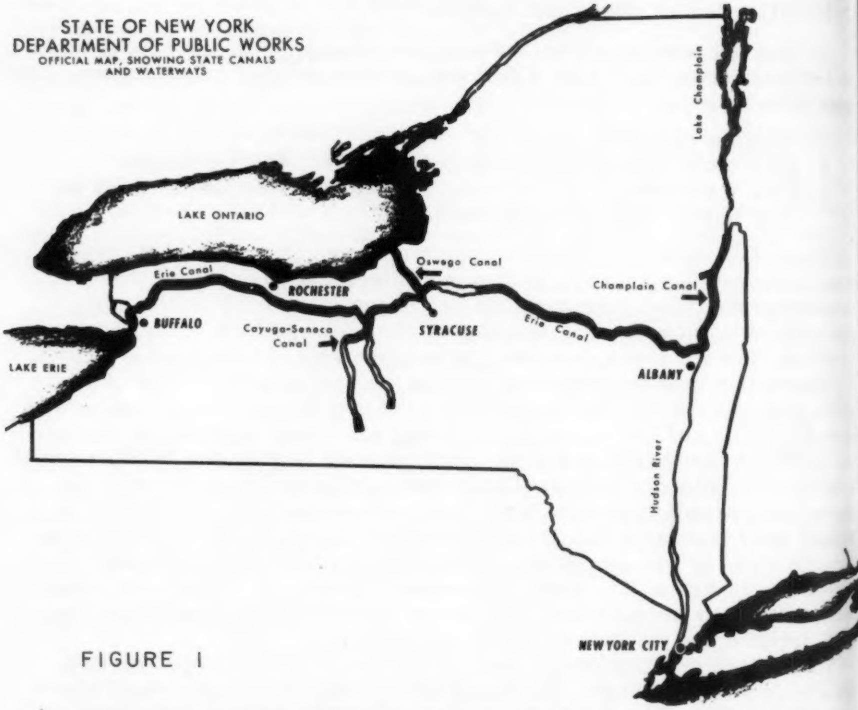

FIGURE I

from the Hudson River to Frankfort. There, it becomes an artificial land cut channel to Lock E-21 at New London west of Rome. At Rome is located the eastern summit level of the Erie Canal. Water for navigation is let into this level from canal reservoirs, which are located both north and south of Rome and Utica. From Lock E-21, the Erie makes use of Wood Creek, Oneida Lake and the Oneida River to Three Rivers Point north of Syracuse. From Three Rivers to Mays Point, the Seneca River is used and the Clyde River from Mays Point to Lyons. From Lyons to the Genessee River, just south of Rochester and west to Lockport, the Erie Canal is an artificial land cut channel, often located higher than the adjacent countryside. From Lockport to Tonawanda, on the Niagara River, a land cut channel and Tonawanda Creek are used. The Niagara River is canalized from Tonawanda to Buffalo. This river section is operated by the U. S. Engineers. The Erie Division has 35 locks.

Oswego Division

The Oswego Division, or Canal, uses the Oswego River. It runs from Three Rivers Point, which is the junction of the Oneida, Seneca and Oswego Rivers, to the City of Oswego at the outlet of the Oswego River on Lake Ontario. There are 7 locks on this canal.

Cayuga - Seneca Division

The Cayuga - Seneca Canal runs from Mays Point to Ithaca on Cayuga Lake and Montour Falls, just south of Watkins Glen on Seneca Lake. This Canal

utilizes the Seneca River from its junction with the Clyde River up to and including the two largest Finger Lakes, Cayuga and Seneca. This division has four locks in the canalized Seneca River.

Canal Mileages

The total mileage of each division of the Barge Canal is as follows:

1. Erie Canal from Waterford to Tonawanda - 348 miles.

   Of this mileage 220 miles comprise canalized rivers and lakes and 128 miles of artificial land cut channels.
2. Champlain Canal from Troy to Whitehall - 63 miles.

   The canalized Hudson River takes 40 miles of this total mileage and the artificial land cut section 23 miles.
3. The Oswego Canal is the canalized Oswego River from junction with the Erie Canal at Three Rivers Point to Lake Ontario - 24 miles.
4. The Cayuga and Seneca Canal, which is the canalized Seneca River and Lakes Cayuga and Seneca, runs from the junction of the Erie Canal at Mays Point to Ithaca and Montour Falls - 92 miles.

The total mileage of the New York State Barge Canal System is 527 miles. Of this total mileage, 151 miles comprise artificial land cut channels and 376 miles consist of canalized rivers, lakes and streams.

Canals—Depth and Width

The Champlain Canal, Cayuga - Seneca Canal and the Erie Canal, from Three Rivers to Tonawanda, have a navigable depth of 12 feet in the canal channel and in the locks over lock sills. These canals have a channel width of 200 feet in river sections and 94 feet width in rock sections, they have a bottom width of 75 feet and 123 feet at the water surface in the artificial land cuts.

The Erie Canal, from the Hudson River to Three Rivers Point, and the Oswego Canal have been deepened and widened. This work is slowly coming to completion and is being done with the help of federal funds. It includes the deepening and widening of the channels, deepening of the locks from 12 feet of water over sills to 13 feet, and raising of all fixed bridges to provide 20 feet of clearance above maximum navigable pool level. Of this work, the widening and deepening program is 99 per cent complete. The work of dropping the lock sills one foot is about 80 per cent complete, while the work of raising the fixed bridges is about 55 per cent complete, and presently is at a standstill.

The channel in this so-called improved section of the barge canal has a minimum depth of 14 feet. It is 200 feet wide in river sections and 120 feet in width in rock sections. In the artificial land cut channels, the minimum bottom width is 104 feet and 160 feet at the water surface. Fig. 2 shows some typical traffic on this reach of the Canal.

Bridges Over Canals

Spanning the canals are a total of 309 highway and railroad bridges. Of this total, two bridges are double-leaf bascule, two are single-leaf bascule, one is a plate-girder swing type, and sixteen are lift bridges. The Erie Canal is crossed by 245 bridges, of which sixteen are of the lift type and one is of the

Fig. 2. Eastbound Grain Barge and Westbound Pushed Oil Barge Passing in the Improved Artificial Land Cut Channel of Erie Division, Barge Canal, Near Utica.

double-leaf-bascule type. The Oswego Canal is crossed by fourteen bridges. One of these bridges is of the double-leaf-bascule type, two are of the single-leaf-bascule type, and one is a plate-girder swing bridge. On the Champlain Canal are thirty-four bridges, and sixteen bridges across the Cayuga - Seneca Canal. The maximum clearance under the fixed bridges is 15 feet, 6 inches.

Locks—Size

The barge canal locks are 45 feet wide, 328 feet long between lock gates, and have an available length of 300 feet in the clear in the lock chamber. There are a total of 57 locks on the entire Barge Canal System. Two sets of these locks are tandem locks. The lock gates and valves are operated by electrically-powered machinery. The average time of locking a vessel through the lock is twenty minutes.

Maximum Size Tow

One can see from these physical dimensions that the maximum size of floats (barge and tug, or motorized vessel) that can navigate on the Barge Canal are: length, 300 feet; beam 43 1/2 feet; height above water, 15 feet; and economical draught up to 11 feet in the canalized rivers and lakes, and 9 feet, 6 inches in the artificial land cuts, except 11 feet in the improved canals. These dimensions reduce to a tug and a barge of 3,000 tons capacity (maximum), which is equal to a freight train of 85 tank cars.

Interesting Engineering

The New York State Barge Canal is remarkable from the point of view of construction because of the many types of structures that are used. They include fixed dams, movable-bridge type dams, taintor-gate and sector-gate type dams, guard gates, siphon type spillways, lift bridges, retention dams, aqueducts and a siphon-operated lock—to name a few of the structures. The job of planning the Barge Canal involved a great deal of engineering. Much of the engineering fell within the field of new design. Much was in the field of adapting old principles to new conditions. In the light of comparative engineering knowledge, there is no doubt that New York State, in building the Barge Canal, has accomplished as great an engineering feat as was accomplished almost a century before in the building of the Grand Old Erie. Fig. 3 shows repairs in progress on a lock wall.

Fig. 3. Derrick Boat Tenkenas Making Repairs to Bullnose, Lower End River Lock Wall, Lock 10 Erie

Water Sources

The Barge Canal is distinctively a river canalization project. The natural flows in the canalized streams and rivers supply the canal water needs to a large degree. The Champlain Canal summit level at Smiths Basin is supplied by means of the Glens Falls Feeder. The feeder takes the water from the Hudson River west of Glens Falls. The water supply for the Champlain Canal is more than ample, as the entire watershed of the Hudson and Sacandaga Rivers, west and north of Glens Falls, is available for this purpose. Regulated Cayuga, Seneca and Oneida Lakes, with their tremendous capacities, provide the water requirements for navigation on the Oswego and the Cayuga - Seneca Canals, and on the Erie Canal, from Mays Point to Three Rivers, and from Lake Oneida to Three Rivers. The western section of the Erie Canal had an unlimited water supply in Lake Erie by way of the Niagara River. With the redevelopment of Niagara Falls for power, this source will be limited. However, sufficient water will be available for all navigation needs on this portion of the Barge Canal. Here the flow is through the canalized Tonawanda Creek and the land cut to Lockport; through and around tandem Lock E-34/35 and along the famous Sixty Mile Level to the Genesee River at Rochester. From Rochester, some of the surplus Sixty Mile Canal Level water, together with some Genesee River water, is used for power development at Rochester and to supply the Erie Canal levels east of the Genesee River up to Lyons, and, as a matter of fact, farther eastward, up to Mays Point, where the canalized Clyde River joins the canalized (Cayuga - Seneca Canal) Seneca River.

On the eastern section of the Erie Canal, the natural stream flows must be augmented by canal reservoirs. For this purpose, most of the old Erie Canal reservoirs and feeders, that were built to supply water to the old canal, were retained to supply the Barge Canal summit level at Rome. In addition, large

new reservoirs were built under Barge Canal contracts to supply this level. Delta Reservoir, located just north of Rome, supplies the summit level by means of the old Black River Canal, while Hinckley Reservoir, which is located east of Rome and north of Utica, feeds this level from West Canada Creek by way of Nine Mile Creek and Feeder. The flow in West Canada Creek, south of the Nine Mile Creek Feeder junction, enters the canalized Mohawk River, east of Frankfort and Lock E-19.

The Barge Canal planners were liberal in their provisions for an adequate water supply for navigation on the canals. The water provisions are ample for moving ten million tons of traffic on the Erie during a canal navigation season, even under severe drought conditions.

Terrain Problems

Some of the terrain features that had to be overcome in building the Barge Canal are interesting. At the eastern end of the Erie Division, an artificial land cut channel was built to carry the canal from the Hudson River to the Mohawk River around Cohoes Falls, which are located in the Mohawk, just west of its confluence with the Hudson. In this short section of the canal, 1-1/2 miles long, there are located five locks, which have a total lift of 169 feet. The lifts in these locks range from 32-1/2 feet to 34-1/2 feet. It can be safely said, even today, that this group of five locks form the greatest series of high lift locks in the world.

This 1-1/2 mile section of canal was the most interesting in its planning, design and construction.

The engineers pondered, should the design be a combine of locks or a series of separate locks. Operation considerations, safety requirements, and the intricacies of design of the many other structures and facilities in this short stretch of canal, determined the use of separate locks with short navigation pools between locks. This required the design of special bypasses around the locks, and retention pools with self-regulating dams. These pools must accommodate the spillage of the locks without causing overflowing of the canal banks, while, at the same time ponding sufficient water for filling the locks without appreciably lowering the short navigation pools.

Further complications to the engineering design were caused by the inclusion, at the upper end of this short canal section, of a long, high dam, with a powerhouse at each end, which forms Crescent Lake. A deep rock cut, about a half mile in length, had to be provided from Crescent Lake to Lock E-6. For safety reasons, two guard gates, each with a taintor gate bypass, had to be designed for location in this rock cut. Special navigation approach walls between locks were provided in the short navigation pools between locks. A design for high retaining walls was necessary at the retention pools between the locks, and from Lock E-2 to the Hudson River. Two railroad, and several street and highway crossings had to be made.

In addition, plans included into this short section of canal a dry dock and canal shop at Lock E-3, and a canal terminal below Lock E-2. A highway shop was included between Lock E-2 and E-3. Engineering design also included high voltage AC transmission lines from the Crescent Dam powerhouses to the guard gates, two lock powerhouses, and the shops and dry dock. Transformer stations had to be provided. The two lock powerhouses had to be designed to convert alternating current to direct current, and the DC transmitted to each of the five locks, one guard gate, and to the dry dock and shops for

operation of DC machines and equipment at those locations. Approach highways to the locks, shops and dry dock had to be provided. Several bridges and culverts to carry these approach roads over spillways and the canal had to be designed.

It is no wonder, therefore, that, during construction, and after completion of this short 1-1/2 mile section of the canal, it was the Mecca for visiting engineers, not only from this country, but from many parts of the world. Even today, many engineers and engineering students visit this section of the canal system.

The western section of the Erie has the famous 60-mile canal level. This level extends from the Genesee River at Rochester to Lockport. The difference in elevation of the bottom of the canal prism at the two ends of this length of the Canal is less than two feet. The slope of the channel is slightly steeper from Lockport to Medina than from Medina to Rochester. This slight change in slope provides for surplus canal water at Media at Old Orchard Creek for hydroelectric power.

Long sections of this 60-mile level lie above the level of the surrounding countryside. In some locations, the Canal is more than 50 feet above the adjacent terrain. In this level are a number of lift bridges, an aqueduct, which carries the canal over Oak Orchard Creek, and a culvert, which carries a county highway under the Canal. To protect the adjoining countryside from flood damage, should the canal banks fail, numerous guard gates have been installed at fixed distances one from the other. Their locations, in general, are near access roads and the lift bridges so that they can be conveniently closed by the lift bridge operators in case of bank failure. To further safeguard the countryside, bank watchmen patrol the banks and watch for seepage, leaks and boring rodents. Figs. 4 and 5 show typical guard gates that are located in long sections of artificial land cut channels where such channels enter or leave river sections.

Oak Orchard Creek at Medina presented a design problem to the engineers. Here, the bed of the creek changed abruptly from a shallow channel to a deep gorge. Here, also, the terrain conditions were such that the old canal practically made a sharp circle to pass around the gorge. Good alignment for the Barge Canal, to permit safe navigation for 1,000-ton and larger barges, demanded a much straighter alignment, and, therefore, the elimination of the old circuitous route. The need for good alignment at this location gave rise to more studies and plans than were required for any other short stretch of the canal.

One of the studies called for a concrete arched aqueduct to span the gorge. This very wide, single span structure, with its long span and enormous dead and live loadings, was startingly unique, as it was the largest and most ambitious single-span concrete structure ever planned up to that time. For reasons obvious to us today, neither this plan nor the many other plans were carried out. Instead, the Barge Canal follows the route of the old Erie Canal. To alleviate the difficulties of the sharp curvature to navigation of this short section by 1,000-ton and larger floats, an unusually wide channel was constructed. This called for very long stretches of concrete canal walls. Since the Canal borders the gorge very closely, some of these walls are exceptionally high and their design presented a challenge to the engineers, which resulted in an unusually interesting design.

Fig. 4. Herkimer Guard Gate

Locks

The most important structures on the canals, of course, are the locks. The locks are built of concrete throughout, both the side and cross walls and the floor, except at a few locations where good rock formation was encountered, and the concrete floor was dispensed with.

Concrete

At the time of the building of the Barge Canal, controls for concrete, as used today, were unknown. Much native sand and gravel, found near the location of the locks and other structures, were used. The present condition of the canal structures reflect, to a remarkable degree, the quality of the materials used in the concrete. Some structures, and portions of others are as sound today as the day they were built. Other structures deteriorated and required considerable rehabilitation. However, it must be said that the canal structures prove the utility of concrete for permanence.

Lock Walls

The side walls of the locks are 5, 6 or 7 feet wide at the top, according to local ground conditions. At the river locks, the top of the river lock wall was increased to 12 feet. The walls vary in height and bottom width with the lift of the lock and certain local conditions. The lifts in the 57 locks vary from 6 feet to 40-1/2 feet. Both the lift and the fluctuation between normal and high navigable pool levels governed the height of the side walls of the locks. These walls vary from 28 feet to 61 feet, with an extreme height of 80 feet at the high lift lock E-17 at Little Falls. Wherever rock or hardpan foundation was available, the lock was constructed directly on such foundation. Elsewhere, wood piles were driven to support the entire lock. Most of the locks on the New York State Barge Canal are so supported.

Lock Culverts

Each side wall of the locks contains a culvert for filling and emptying the lock. The invert elevation of these culverts is the same as the floor of the lock. Ports in the lock wall near the floor of the lock connect the culverts with the lock chamber. These culverts vary in size. The dimensions are 5 by

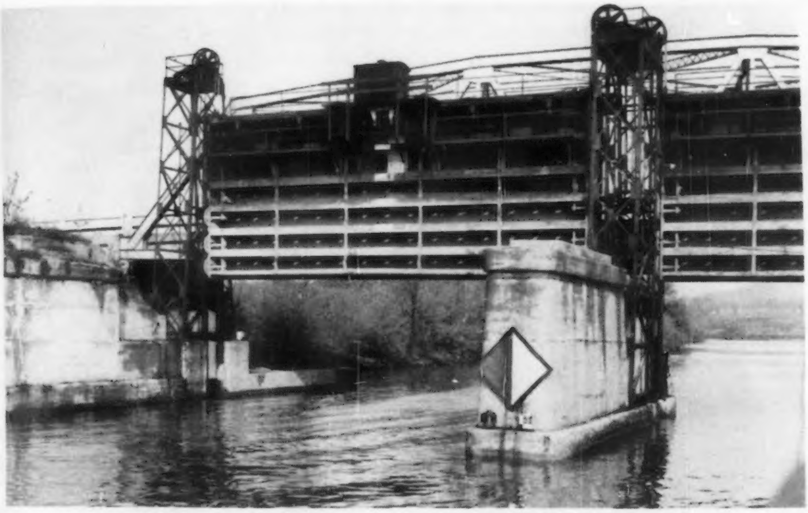

Fig. 5. Typical Guard Gate. Note the Girder Construction of the Gates. The Gate Face Plate is Located on the Upstream Side of the Gate.

7 feet for locks with lifts up to 12 feet; 6 by 8 feet for lifts from 12 feet to 23 feet; and 7 by 9 feet at the locks with lifts of more than 23 feet. The 5 by 7 foot culverts are connected with the lock chamber by 16 ports, eight in each side wall. The number of such ports was increased to 22 for the 6 by 8 foot culverts, and to 28 for the 7 by 9 foot culverts. The ports are either circular or square. The square ports were used in the locks built last. The area of opening of these ports is 7-1/2 square feet each. Water is admitted into and discharged from the lock equally by both culverts to provide for a smooth rise or drop of a float in the lock.

In addition to these lock culverts, 33 locks each have a power culvert built into one of the lock walls. This power culvert is located over the lock culvert. The power culvert carries water from the upper end of the lock to the location of the small lock hydroelectric powerhouse. Here the water is used to develop up to 75 kilowatts of direct electric power for the operation of all lock machinery.

Lock Gates

The lock gates are of the mitering, girder type, carrying the principal load as beams. There are two gates or leaves at each end of the lock. The gates are built of steel and have a single skin plate. The girder sections have a depth of 2 feet, six and a quarter inches, and are spaced on three feet, one quarter inch centers for the deep lower gates, and on three feet four and one-quarter inches centers for the upper gates. In general, the top two girder sections of all gates are spaced about four inches more than the spacing mentioned. Fig. 6 shows Lock 2 on the Erie Canal with the lower gates opening.

The gates are provided with white oak quoin and toe posts. Until several years ago, the gates mitered against 12" x 12" white oak timbers anchored to

Fig. 6. Lock 2 Erie, Lower Gates Opening Completing Down Locking Tug and Barge with 2,500 Tons of Linseed Oil for New York City

the concrete lock gate sill. These heavy timbers acted, both as a seal against leaks between the gates and the sill, and as a gate sill. This timber type of sill was difficult to maintain against leaks between it and the concrete sub-sill. Maintenance personnel devised a sill of concrete and a heavy 10-inch steel angle. To seal against leaks between the gate and the sill, a 4" x 8" white oak timber sealing strip is fastened to the bottom girder of the gates, which bears against the sill.

The quoin posts have cast steel sockets attached to their bottom ends. The gates swing on cast steel pivots, which are concreted into the sill at the hollow quoins. Adjustable steel anchor rods support the gate at the top near the quoin and provide for proper positioning and hanging of the gates. In the closed position, the white oak gate quoins bear against cast steel hollow quoins, which are set in the corner of the gate wall recess. The white oak toe miter timbers on each gate leaf bear one against the other when the gates are closed. Under pressure of water in the lock, these contacts form tight water seals.

Lock Gate and Valve Machinery

The lock gates are opened and closed by a 7-horsepower direct current motor. The power from the motor is applied to a steel spar with rack, which moves in a horizontal plane, by means of ring and bevel gears. The opening and closing time of the gates is about one minute.

The lock valves, which regulate the flow of water into the lock culverts, are suspended by means of heavy wrought iron power chains in the valve wells. Two chains support each valve and its counterweight. These chains pass over chain cup wheels, which are mounted on a shaft connected to an electric motor

by means of a gear train. The 5-foot by 7-foot valves and the 6-foot by 8-foot valves are operated by a 3-horsepower direct current motor, while it takes a 7-horsepower motor to operate the 7-foot by 9-foot lock valves.

Both the lock gate and valve operating machinery are provided with limit switches and overload devices. Signal lights indicate the position of the valves. In case of failure of the gate or valve operating machinery, the gates or valves can be operated manually. The gearing ratios in the train of gears are such that two men, working a sweep, can open or close the valves or gates with ease.

Operation and Control of Locks

Each lock is provided with a traffic light at each end. These traffic lights indicate by a red signal to an approaching float that the lock is not ready and for the float to tie up at the lock approach wall. When the lock is ready to receive a float, a green light is shown. Electrically-powered capstans are also provided at each lock. These are located on one side of a lock, one capstan at each end. The capstans were provided to control the movement of boats along the approach walls and to tow them into and out of the lock chamber. Today, the capstans are seldom used since only a few of the so-called "double-locking" fleets operate on the canals. The present canal floats, comprised of a tug and a 2,000- to 3,000-ton capacity barge, do not need this help.

Electric Power for Locks

When the New York State Barge Canal System was built, the electric power at the 57 locks for operating the gate, valve and capstan machinery, the lock lighting and heating, the signal lights and other electrical devices, was supplied by 33 hydroelectric stations, 11 gasoline-electric stations, 2 transformer with AC to DC converter substations, and 1 motor generator station. Six locks are supplied with direct current power from near-by locks with hydroelectric stations, and three are supplied from near-by locks with AC to DC converter stations.

Each lock hydroelectric power station, operated by canal water, has two, low speed, vertical-shaft turbines installed. These turbines are connected directly to vertical-shaft generators which supply direct current at 250 volts. At four low-lift locks, where only a small head of water is obtainable, horizontal shaft generators are installed. 75-kilowatt waterwheel-generator units and booster sets are installed at the locks, which supply electric current to adjacent locks (two miles or less away), while 50 kilowatt units are installed at all the other lock hydroelectric stations. The gasoline-electric stations were each provided with two 25-kilowatt, 250-volt direct current generators directly connected to gasoline engines, designed to run at a speed of 600 rpm.

Operating experience has shown that it was more economical to purchase alternating current power and convert it to direct current by means of a motor-generator set, than to generate it at the gasoline-generator stations. For this reason, 15-kilowatt motor-generator sets have been installed at all of the gasoline-electric stations. The gasoline-electric units are now used as stand-by units.

Lock 17

Recently, Lock 17, Erie Canal at Little Falls, has been completely rehabilitated. Because extensive work had to be done at the hydroelectric station and the power-culvert of this lock, it was determined that it would be more economical to purchase power. It was further determined that excellent operation of the lock machinery could be obtained with alternating current. As a result, new alternating-current gear motors were installed in place of the direct-current motor drives and the speed-reduction gear trains. Since this lock has a lift of 40-1/2 feet, it is the only lock, except the two guard locks at the Genesee River and the Utica Harbor Lock, that is provided with a lift type of gate. This 125-ton lift gate, with its associate concrete counterweight, is controlled in its operation by selsyn motors. The operation of this lock during the 1958 navigation season on the canals indicates that alternating current is well suited for such an application.

Lock 8 Siphon Control

Lock 8, Oswego Canal, located in the city of Oswego, is the only one of its kind on the Barge Canal. This syphon lock was the first of its kind to be built in this country and the largest employing the syphon principle. This lock, in design, is similar to the regular locks except in the design of the lock culverts. The upper and lower ends of the lock culverts are curved up to form necks. These necks, or siphons rise a little above the highest water level. They are also tightly shut off from the outside air except through operating pipes. Large tanks are built into each wall at the upper end of the lock. These tanks are connected by piping with the upper and lower pool levels, and with the upper and lower siphons in each wall. The tanks, siphons and piping must be maintained airtight. To fill or empty a lock, the siphons must be started and operated by vacuum and air. This is accomplished by first filling the tank with water, then closing the tank inlet valve and opening the outlet valve. This operation results in a body of water desiring to escape from the tank, but being prevented from doing so by the vacuum it produces by its volume and weight. On opening the air valve in the pipe line from the siphon to the tank, the air in the siphon rushes into the vacuum and lifts the water into and through the neck of the siphon. Using both the upper or lower siphons, the Oswego Lock 8 chamber, having a lift of 11.1 feet, can be filled in five minutes and emptied in six minutes, respectively. Theoretically, the operating power is self-renewing and, except for air leakage, lockages could be made by merely manipulating the 4-inch valve and the tank valves.

The ever present maintenance problem of keeping the tanks, siphon and much of the piping completely air-tight led to a modification of the system. Vacuum pumps and simplified piping were installed. The air in the necks of the siphons is now exhausted through control valved piping connected to the pumps.

Dams

Aside from the locks, the dams are the most important structures on the Barge Canal System. Since the Barge Canal was a river and stream canalization project, 40 dams were required. To meet specific and widely different stream and stream flow conditions, several types of dams were designed.

Fixed Dams

Of the fixed dams, the four largest and most interesting are the Hinckley and Delta reservoir dams, and the two dams across the Mohawk River, east of Schenectady at Vischers Ferry and Crescent. Most of the fixed dams have some moving parts incorporated in their construction for the purpose of regulating the navigation pool levels and passing freshet waters.

Crescent Dam, which derives its name from the name of a near-by village, is located just below the entrance into the Mohawk of the short land-cut channel with its five locks from the Hudson River. The dam is curved in plan and of the gravity type. It does not depend on its curved form for stability. This structure is really two dams with a rocky prominence intervening. One section of the dam is across the former river channel, while the other section was built across low land, which, after completion of the structure, was submerged. The structure has a total length of 1,922 feet, and sweeps through nearly a semicircle, having a 700-foot radius. Across the front of the portion of the dam that was built on the low land, a third dam was built. This dam has a low elevation and its purpose is to maintain a pool to serve as a water cushion to break the fall of the water spilling over the crest and prevent erosion of the rock at the foot of the main dam. The pool back of the dam has been raised 28 feet above the former river level and forms a virtual lake. During the ceremonies attending the opening of this section of the Erie Canal on May 15, 1915, Governor Whitman formally christened the body of water "Lake Crescent."

This dam is 42 feet wide at the base and 11 feet, 5 inches wide at the top. The crest is 39 feet above the apron. The apron has a width of forty feet. The total masonry content of this dam is 54,360 cubic yards.

At the western end of this dam, farthest from the canal center line, is situated one of two large hydroelectric plants that were built as part of the canal system. Each of these plants has two 3,000-kilowatt, 2,400-volt vertical-shaft waterwheel generator units. This Crescent station supplies electric power to the two guard gates at the top of the Waterford flight of five locks, the five locks, the large canal shop and canal dry dock, and to the near-by highway shop. The surplus power is sold to a public utility under a 25-year contract.

About ten miles above the Crescent Dam is located the Vischers Ferry Dam with Erie Lock 7 at its southern end, and the Vischers Ferry canal hydroelectric plant at its northern end. This structure is similar in design, except alignment, to the Crescent Dam. The dimensions of the two dams differ very little. The site chosen for this dam is near the village of Vischers Ferry and was one having two river channels encircling an island of considerable size, which had steep shores and a rocky plateau-like top about twenty feet above the river surface. Each of the channels was dammed. A third section, built across the island, connected the two river sections of the dam to make a continuous crest of nearly 2,000 feet. Each section of this dam is straight. However, their location in the river channels and across the island is such that the plan of the whole structure is roughly that of a reversed "Z".

This dam contains 90,000 cubic yards of masonry, and its crest is 36 feet above the apron. The hydroelectric plant at its north end is identical with that at the Crescent Dam. This plant supplies electrical energy for the operation of Erie Canal Lock 7. The surplus energy is sold to a public utility under a 25-year contract.

The two large canal reservoir dams, Delta and Hinckley, were built for the purpose of supplying water to the Rome summit level of the Erie Canal. By restoring an ancient glacial lake, five miles north of Rome, Delta Reservoir was formed. The Delta Dam is about 1,100 feet long with a spillway, 300 feet long in its center. Its height is 100 feet from lowest foundation to crest. It forms a lake with an area of four and one third square miles at crest with a maximum depth of 70 feet, an average depth of 23 feet, and a capacity of 2,750,000,000 cubic feet.

Hinckley Reservoir lies on West Canada Creek in the foothills of the Adirondack Mountains. The dam of this reservoir has a total length of 3,700 feet, which forms a lake 4.46 square miles in area with a maximum depth at crest level of 75 feet, an average depth of 28 feet and a total capacity of 3,445,000,000 cubic feet.

Bridge Type Movable Dams

Of the movable dams, the Mohawk bridge type dams are very interesting. There are eight of these dams, ranging from 370 to 590 feet in length and one smaller one, all located on the Mohawk River, west of Schenectady at Erie Canal Locks 8 to 15, inclusive. Small editions of these dams are also located on the Mohawk River at Rocky Rift and Herkimer, and on the Seneca River at Mays Point. The depth of water back of these dams from crest to sill varies from 16 to 20 feet.

The Mohawk River, because of its location in a fairly steep valley, and because of the type of streams that flow into it, is subject to very fast rise and fall and to severe ice jamming during the winter. Because of these conditions, the canal designers concluded that some type of movable dam must be used to control floods and ice outflow to the extent of restoring natural stream conditions. Their studies led them to adopt the bridge-type movable dams for their existing locations because they would perform their required functions at the least cost. Maintenance experience has proved that the cost of maintaining these dams is not much more than the cost of maintaining a similar-type highway bridge. Fig. 7 illustrates the bridge-type movable dam.

The interesting feature of this type of dam is that the structure is a bridge with abutments and piers. It is designed to take a tremendous lateral pressure. These structures are provided with a dam foundation, a sill and an apron in the stream bed between the piers. On the downstream side of these bridge structures, special hinge pockets are provided every 15 feet along the lower chords of the trusses. In the dam sill, cast iron shoes are set in at locations directly under the hinge pockets. Dam gate uprights are supported by pins at the hinge pockets and, in the down position, the toe of the gate upright rests against the cast iron shoes. These dam gate uprights are girder sections, which have a straight flange on the upstream side and a variable depth ranging from 1 foot, 5 inches at the hinge end to 1 foot, 10 inches at the toe with the maximum depth of about 3 feet occurring about one third of the girder length above the toe.

Gate pans, supported by wrought iron chains, slide along adjacent uprights to form the dam. Two to three such gate pans, one above the other, form the dam to the required depth at each location. The upper gate pans may be full gates or two half gates. The location of these dams along the Mohawk River determined this choice for the reason that the upper gate pans of all of these bridge-type movable dams are raised or closed as required to regulate the navigation pool and flow in the river under high water conditions.

Fig. 7. The Mohawk Bridge-Type Movable Dam. Note the Hinged Gate Uprights and Wrought Iron Chains for Lifting the Gate Pans and the Gate Uprights. An Electric "Mule" Shown on Bridge Raises the Gate Pans Individually and the Uprights in Pairs. A section of Catwalk is Attached to Each Pair of Uprights.

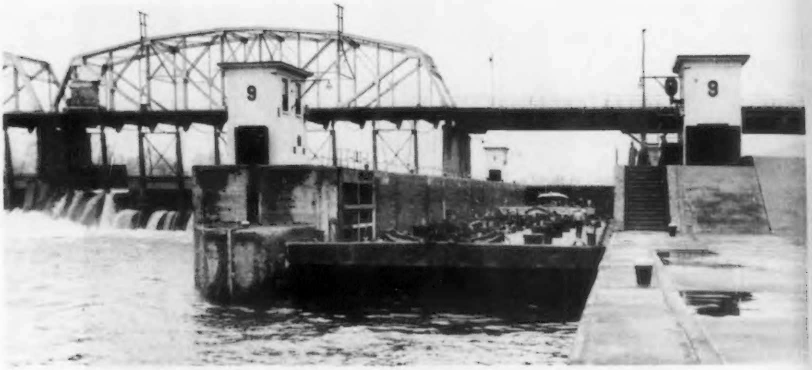

Fig. 8. Downstream Side of Movable Dam. Note Raised Gate Pan to Pass High Water. The Leakage Shown is Occurring Between Gate Pans.

The gate pans and the dam gate uprights are raised or lowered by means of mobile electric winches through wrought iron chains, supported by properly placed sheaves. During the close of navigation, the gate pans and the uprights are raised to provide a clear river channel at these structures. Fig. 8 shows this operation.

Taintor Gate Type Movable Dams

The taintor-gate type of movable dam is used very widely on the New York State Barge Canal. This type of dam is used in a wide variety of ways; as a whole dam, as a regulating section in conjunction with a fixed dam, as a gate to fill a notch in a fixed dam, as a by-pass gate at a lock or guard gate, and as a crest across the top of a low fixed dam. Figs. 9 and 10 illustrate this type of dam.

An unusually large dam of this type has been incorporated with the fixed dam in the Hudson River, just above Waterford at Lock No. 1 of the Champlain Canal. This dam has six gates each 50 feet wide and a vertical height of 17 feet above the sill. These gates serve as a regulating section in connection with the fixed dam. A single gate of longer span is located adjacent to Champlain Canal Lock 12 at Whitehall. There, it forms a movable crest on a low fixed dam. This taintor gate dam is operated from a bridge over the stream at this point, which also carries a highway across the stream. The clear length of this gate is 90 feet. Other conspicuous examples of taintor gate movable dams are those at Cayuga, adjacent to Cayuga and Seneca Canal Lock 1, and in the Oneida River at Caughdenoy. Here the taintor gate dams constitute the entire structures that act as regulating works for Cayuga and Oneida Lakes, respectively.

Special Court Street Dam

A unique dam was constructed in the Genesee River at Court Street in Rochester. This dam is a combination of bridge and taintor gate types. This dam provides the pool level for the Genesee River crossing of the Barge Canal south of Rochester. In the usual taintor gate dam, the sector gate swings on

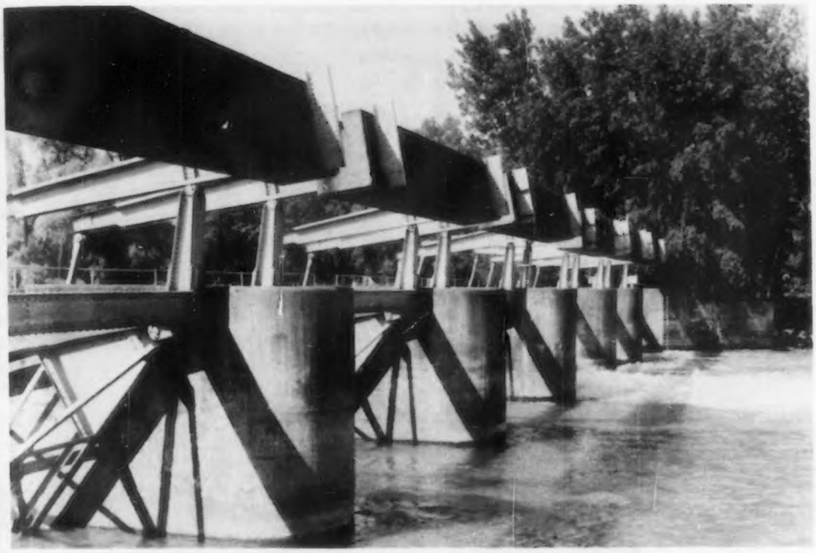

Fig. 9. Downstream Side Caughdenoy Taintor Gate Dam Showing the Hinging of the Gate at the Pier and Gate Counterweight.

pivots located above the stream. When the gate is lifted, usually by a fixed pinion gear rolling on a gear rack affixed to the gate, the discharge occurs between the gate and the gate sill. In the adaptation of Court Street, Rochester, Dam No. 10, the sector gate is pivoted at the stream-bed level and the gate drops into a recess in the masonry instead of being raised. Water, therefore, passes over the gate instead of under it, and presents an unobstructed crest to the water, regardless of the position that the gate is in,—fully open, fully closed, or somewhere in between. Such a gate becomes a clear and free adjustable spillway.

It was for this reason that this adaptation of the taintor gate principle was used. The Genesee River carries tremendous debris and flood wood. To prevent the debris from lodging in the ribs, radius members, and other reinforcing and supporting steel of the gate, a decking is provided on the radius members from the face of the gate to its pivot. This decking forms an inclined plane down which the water may flow and the debris pass with ease. The slope of this inclined plane can be changed by simply raising or lowering the gate, which is accomplished hydraulically.

Siphon Spillway

Of the many other types of structures that can be found on the New York State Canals, one deserves some comment. This structure is the siphon spillway. Such spillways are especially useful at those navigation levels where there is no room available for the usual waste weir, or where it is impossible to install a long overflow spillway, but where it is necessary to automatically regulate the pool-level elevation within certain fixed limits. It was the presence of such conditions which led to the design and introduction of such a

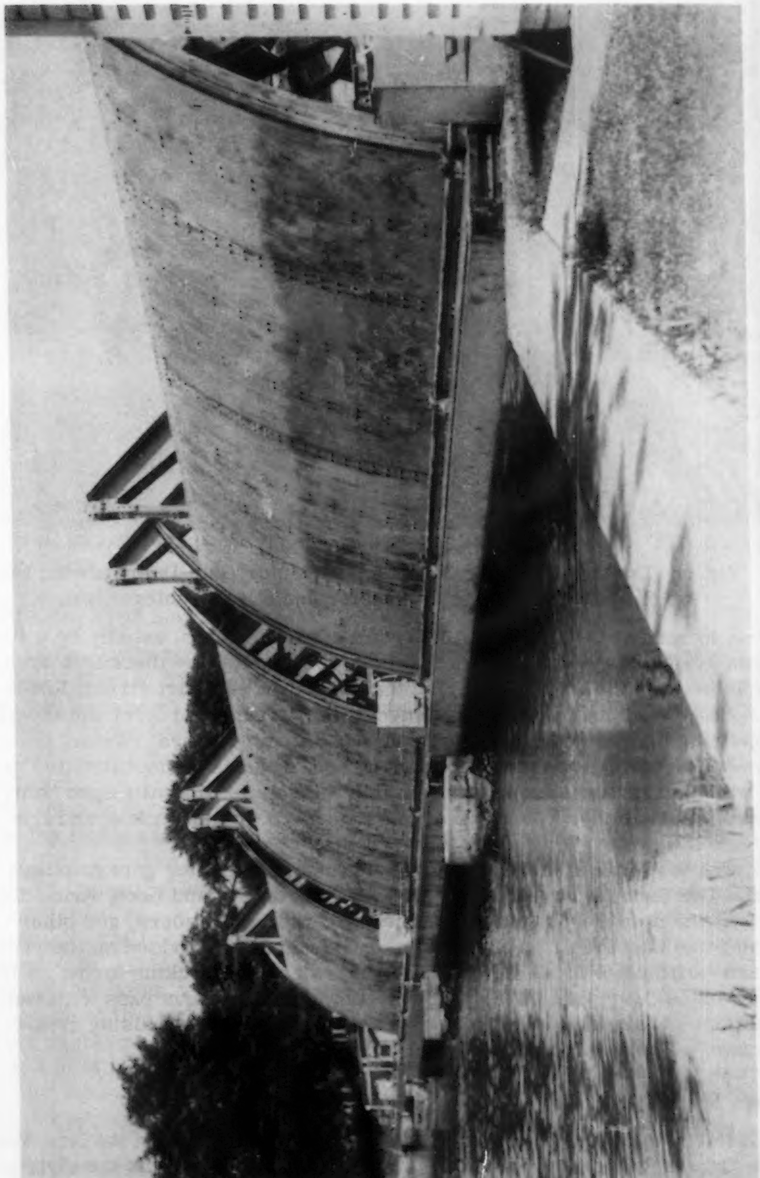

Fig. 10. Upstream Side Waterloo Taintor Gate Dam. Three Gates Opened to Pass Flood Waters.

new structure. It is believed that the siphon principle was used for the first time on the Barge Canal to create such a spillway of any considerable size. The siphon action is entirely automatic in both the starting and stopping of the flow. These types of spillways appear to be able to accomplish as much as the regular spillways, three to five times as long, providing they are kept cleared from debris and leaves.

Such spillways have siphons built into a concrete wall which externally differs little from any wall that might be built to separate two bodies of water having different surface levels. The siphon is a cavity in the wall. Its inlet is placed well below the surface of the pool to be regulated. Its outlet is placed as low as possible below the surface of the water in the stream that carries away the flow. The crown of the siphon rises to the elevation at which it is desired that the discharge of water shall begin. The bottom of the crown is at the elevation where it is desired that the flow of water shall stop. At this level air vents pierce the wall to permit outside air to enter the siphon. The siphons start working when they fill up. When the regulated pool level has been drawn down to the desired elevation, air enters the siphons through the air vents and the flow ceases.

The siphon-type spillways are very effective, provided they are kept open. Old engineering records indicate that such spillways were more economical to build than the normal waste weirs for handling the same quantity of water. Canal maintenance experience, however, points to the requirement of considerably more maintenance to keep these spillways free and clear so that they function properly. For this reason, one or two such smaller spillways have been sealed and conventional spillways built instead.

Canal System Operation and Maintenance

The New York State Canals are normally opened to navigation by the second week in April and closed to navigation usually at the end of the first week in December. Excluding very short delays to navigation by high water, the canals are open to navigation approximately 8 months each year.

The Erie, Champlain and Oswego Canal locks are operated 24 hours per day and seven days a week during the entire navigation season. The Cayuga - Seneca Canal locks are operated from 8 o'clock in the morning to midnight seven days a week during the entire navigation season. Commercial floats, desiring to lock through the Cayuga - Seneca Canal locks between the hours of midnight and 8 o'clock in the morning, must make arrangements for such lockings with the Canal Section Superintendent's Office at Lyons.

Aids to Navigation

The canal channels in all stream, lake and river sections are marked with lighted buoys. In the stream and river canal sections, the buoy lamps are kerosene type and remain lighted continuously. In the lake sections, the buoy lamps are battery-operated and are provided with sun relays, automatic lamp changing and flasher mechanisms. These lights operate only at night and on dark and stormy days, when the light intensity is only of the order of twilight. Large 5-inch diameter, plastic-type reflectors are mounted on each buoy above the buoy lamp as an auxiliary night navigation aid. These reflectors are picked up by float searchlights and indicate buoy and channel locations should the buoy lamps be out.

The artificial land-cut channels on the Erie Division are provided with kerosene buoy lamps supported on steel stakes installed along the canal berm to provide an aid to navigation during the night. The artificial land-cut sections on the other canals do not have such navigation aids. Large plastic-type reflectors are, however, mounted on all bridges crossing the artificial land-cut channels and at all curves in the land-cut sections of the Champlain Canal to delineate the channel at these locations by reflected light from searchlights at night.

Maintenance Organization

As previously pointed out, the New York State Canals have structures of all types. They also include access roads, buildings, drydocks, shops and floating plant, such as dredges, tugs, derrick boats, etc. The operation and maintenance of the canals, therefore, requires many engineering and technical skills. The maintenance organization must be able, versatile, flexible, willing and imaginative: to meet emergencies of operation and maintenance; to develop special skills to meet variable conditions which exist; to perform design work; to provide the needed supervision and administration throughout all levels and perform all functions required and set by the Canal Law.

In general, channel and bank protection work is performed during the navigation season. Work on many canal structures and parts of structures is done at this time also, when such maintenance, repair and rehabilitation work does not interfere with navigation. The work at locks, dams, guard gates and other structures, which would interfere with navigation, is done during the close of navigation from early December to early April. This work generally requires cofferdams to dewater the work areas. See Figs. 11, 12 and 13.

The location of the major maintenance, repair and rehabilitation work determines whether floating plant, land-based equipment, or both are used. Floating plant equipment, that is not used in winter work, is laid up in drydock for overhaul and repairs.

Canal Shops

Throughout the year the 8 canal shops perform all the machine and repair work required for maintenance of automotive and construction equipment, floating plant and lock and other structure machinery. These shops perform welding, carpenter, machine, blacksmith, electric and mechanical shop work. They lay out, fabricate and construct pontoons for dredge pipe, lock valves, movable dam gates, small scows, dredge pipe, buoy boats, etc. They rebuild dredge pumps, machine canal castings, cut gears, shape miter and quoin timbers, and perform a multitude of other construction, fabrication and machine functions that cannot be found in any shop of comparable size.

Maintenance by Operations Personnel

Not all maintenance work is done by canal maintenance personnel. When navigation on the canals is closed, the lock operating personnel perform the major repairs and overhaul of the various electrical and mechanical machinery and equipment at their locks. Waterwheels and generators are repaired, cleaned and overhauled. Electric motors are taken apart, cleaned and reinsulated; commutators dressed; field and armature windings varnished and baked and bearings replaced where needed. All panel boards are cleaned and

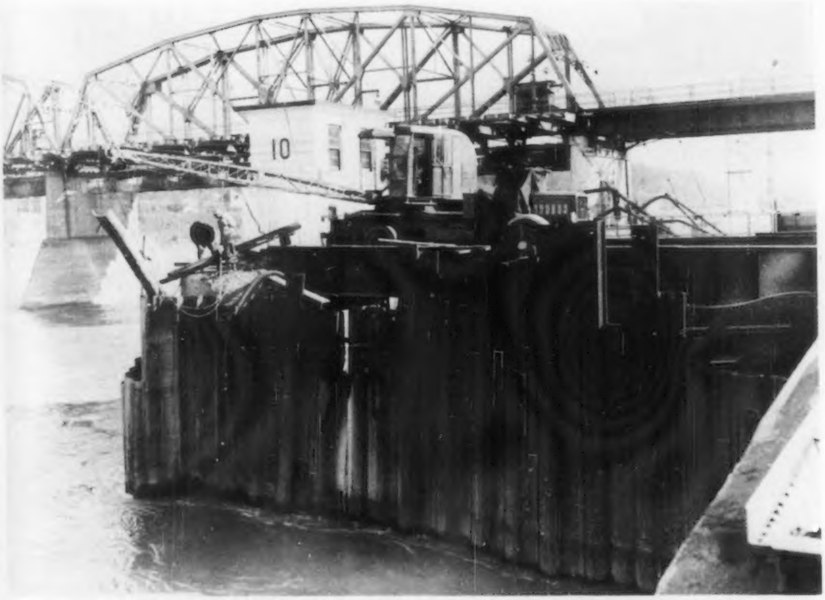

Fig. 11. Major Repair Work is Performed in the Winter. Note Special Type of Deep Steel Sheetpile Cofferdam in Foreground. Truck Crane on Temporary Bridge Placed Over Lock. Note Pump Outlet Hose Back of Truck Crane and Movable Dam in Full Open Position.

switches, relays, contactors and instruments are taken apart, cleaned and overhauled. Mechanical equipment, such as shafts, cables, chains, gears, chain cup wheels and bearings, as well as the lock gates and valves and their mechanical operating equipment, are inspected thoroughly for wear, deterioration and faults. Such inspection information becomes the basis for scheduling major repair work as necessary.

It can be seen that the maintenance of New York State Canals is a year round operation in which both the operation and maintenance canal personnel take part.

Economic Considerations

Transportation Benefits

It is a historic fact that bulk commodities, liquid and dry, can be shipped at the lowest transportation cost by water. This is proved by the economic and industrial development of those areas in New York State and Nation, which were fortunate to be endowed with navigable waterways. It is being proven by the billions of dollars, which have been invested in recent years, and which are presently being invested, for new plant facilities and extension of existing facilities along the inland navigable waterways of the Nation.

The New York State Barge Canal System provides such transportation to 85 per cent of New York State's population and their agriculture, business,

Fig. 12. Work Shown Here is at Lock 7 Erie Canal. Badly Cavitated Lock Walls Will be Resurfaced with "Prepact" Concrete. Workmen Chipping 12" of Old Concrete from Walls Prior to Installation of Steel Dowels, Reinforcing Steel, Forms and Pouring etc. of the Concrete. Several Locks Have Been Relined with Wrought Iron Plate. At These Locations the Old Concrete Had to be Removed 12 - 18 Inches Also.

commerce and industry. It also serves the northeastern section of the United States and much of industrialized Canada.

This service that the Barge Canal provides to our State and Nation and to Canada is reflected by the movement of traffic through the canals. The average annual movement is as follows:

1. Per cent of total tonnage originating within the State for delivery within the State 39.98
2. Per cent of total tonnage originating within the State for out of State delivery 10.57
3. Per cent of total tonnage originating outside of the State for delivery within the State. 32.36
4. Per cent of total tonnage originating outside of State for delivery outside of State 17.07

Some 56 different commodities or items of freight were transported on the New York State Canals during 1957, for which complete tonnage records are available. For simpler reporting, many are grouped together in categories or classes of products. For example, gasoline, fuel oil, lube oil, jet fuel, etc., are grouped together in petroleum products. Items such as soda ash, nevolene,

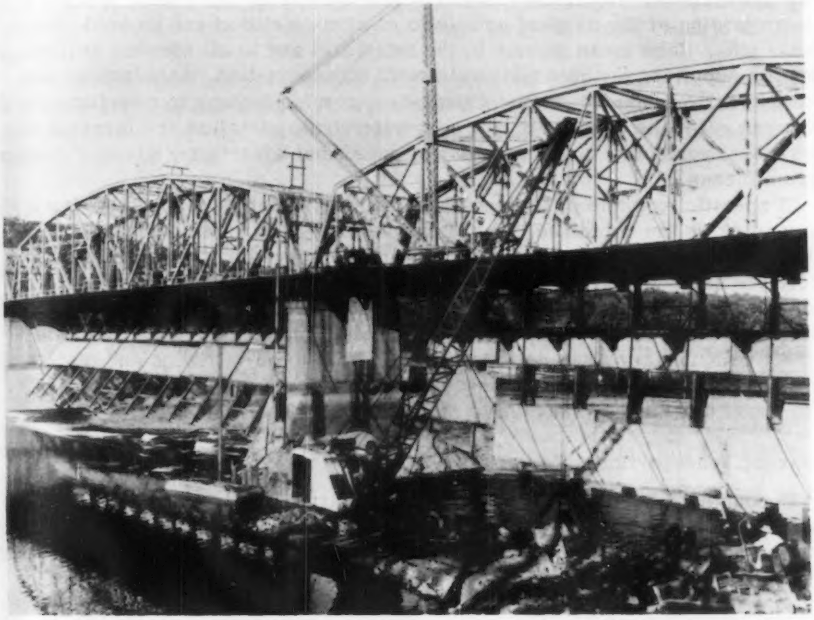

Fig. 13. Reconstruction Work at Lock 9, Erie Canal. Building the Upstream Steel Sheet Pile - Earth Cofferdam Prior to Replacement of Failed Apron and Sill in South Span of Dam. Note Partially Raised Gate Uprights in Middle Span, and the Fully Raised Upper and Lower Gate Pans on Their Respective Uprights.

methanol, etc., are grouped together under chemicals. Other like commodities are grouped into their inclusive product class or category. In 1957, the canals carried 18 products for a total canal tonnage of around 4-1/2 million tons and 130,716,858 ton miles.

The ten leading products that were carried on the Barge Canal were: petroleum products 69.49%; products of agriculture 8.42%; bituminous materials 8.00%; chemicals 3.37%; molasses 2.60%; paper products 1.70%; pulpwood 1.48%; sugar 1.23%; pig iron 1.11%; and fertilizer 0.62%. The volume of these products is indicated in per cent of the total 1957 canal tonnage.

Direct Transportation Benefit

This question often arises, "Just who, besides the oil companies, benefits from canals and inland waterways?" Petroleum products are used by agriculture, business, industry, municipalities, etc., governments, hospitals, churches, homes, trucks, cars, etc. It doesn't take much imagination to realize how much more these materials would cost if they were brought to the consumer by other transport means at two to four times the transportation cost over water transportation. A little thinking on this question will make one realize that a transportation savings to industry, agriculture, etc., in moving

raw and finished materials, etc., can mean a satisfactory profit, and get a lower pricing of the finished article to meet competition and be sold. Sales mean jobs. Jobs mean income to the individual and to all who had anything to do with supplying the raw materials, with transportation, manufacture and sales. Tax on income, corporations, etc., provide income to sub-divisions of government. The benefits of low cost water transportation are far-reaching but often overlooked by the glamour of speed and advertising of other transportation means.

A specific example of transportation saving, which is a direct saving to the consumer, is the computed transportation savings on gasoline and No. 2 fuel oil that was made for the Utica area, which is served by the Barge Canal. Here the savings averaged a trifle less than one penny a gallon. When one considers the millions of gallons of petroleum products, alone, which are consumed in the areas served by the Barge Canal, one realizes that the total direct transportation saving is very substantial to our citizens and their economy. In 1956, the transportation savings in the Utica area on gasoline and No. 2 fuel oil, alone, amounted to more than 1-1/2 million dollars.

Indirect Transportation Benefits

Undoubtedly, the major advantage of the Barge Canal System, and one that is least understood, is its effect of producing so-called water-compelled rates on commodities which are transported by land carriers. Such rates are much lower than they would be otherwise because of the ever present competitive water transportation. There need be no traffic moved on the Canal to effect a transportation saving by such rates to agriculture, commerce and industry, as long as the canals are open for navigation and in good operating condition. This is a historical fact.

Transportation savings resulting from water-compelled rates are an indirect saving, intangible and are difficult to visualize. Data on the volume, source and destination of tonnage moved by land carriers, under such rates, is often difficult to obtain. However, assumptions can be made from the study of the published water-compelled rates and the commodities to which they apply. It appears safe to assume that, for every ton of freight moved on the Barge Canal, 50 tons are moved by land carriers under such rates to prevent water transportation competition, which, if encouraged, could cause a further reduction in these rates.

Economic Survey of Canals

What is this total transportation saving to agriculture, business, commerce and industry in New York State? The answer to this question is given in the "Economic Survey of the New York State Barge Canal." Their survey, prepared in 1950, by Joseph H. Salmon, Management Consultant, New York City, for the New York State Waterways Association, established a benefit-to-cost ratio of 2-1/2 to 1. A return of 2-1/2 dollars to the economy of the State for each dollar spent for operation, maintenance, capital improvements and payment on the bonded indebtedness, can be considered a good investment.

The U. S. Engineers, in their economic studies of the Barge Canal System in 1936 and 1940, concluded that the benefits of the Barge Canal were national in scope. They established, at that time, a benefit-to-cost ratio of better than 2 to 1. On the strength of this survey, the Federal Government has invested

more than 26 million dollars in the improvement of that portion of the Barge Canal that connects the Hudson River at Waterford with Lake Ontario at Oswego.

Port Authorities and many industries regard the canals as vital for their continuing success and existence. The New York State Department of Commerce stresses the importance of the Canal in its program for attracting new industry into the State.

National Defence

The New York State Barge Canal serves National Defense. In the past emergencies, the Canal was utilized to ship vital petroleum and other products to the Northeast. This, of course, was a reversal in the usual direction of flow of this type of traffic. Military landing craft and other boats, built for the armed services on the Great Lakes, moved to the eastern seaboard by way of the canals. At present there are two major air bases located on the canals, at Rome and Plattsburgh, which obtain their bulk necessities by barge. It must be remembered that, in time of national emergencies, rail and motor carriers are taxed to their capacity and water transportation is called on to bear its share of the load.

Non-Transportation Benefits

The Canals benefit the people of the State in other ways besides providing low cost water transportation and water-compelled freight rates. Farmers from Lockport to Rochester use surplus canal water for supplemental irrigation purposes, which means they reap premium quality crops in years of normal rainfall, as well as during years of drought. Surplus canal water is used to generate electric power at Lockport, Medina, Rochester, Fulton, Vischers Ferry and Crescent. This power not only helps industry but the leases for the use of this water for such purpose bring in a substantial revenue to the State. Industries along the canal use this water for industrial processing. Communities along the Canal take water for sanitary and fire purposes. In some communities, it is the only source for domestic water supply. This low cost use of water, without taxing town and city water supplies, is a vital economic factor to industries availing themselves of this facet of canal use.

There is some variance of opinion as to the benefits of the Canal for flood control purposes. It is admitted, however, by flood control engineers that the Canal with its reservoirs and movable dams, properly regulated, contributes significantly for controlled freshet and snow-melt runoffs in the Mohawk River, and from Oneida, Cayuga and Seneca Lakes, and their outflow rivers.

A fairly recent use of the canals, which is developing at an astonishing rate, is in the recreation field. The Canal offers a picturesque and tranquil waterways to the many scenic and historic vacation lands of our State. With the growth of the pleasure boat industry, we can expect this use to increase materially over the coming years. In many areas, the banks of the Canal are dotted with summer camps and boat and yacht clubs. Boat liveries and marinas, healthy small businesses, ply their trade on various levels of the Canal. In the future, our waterways will undoubtedly be aggressively promoted for the tourist and the pleasure boat business.

Transportation Systems are Complementary

Economic surveys indicate that other transportation means have benefited from the economic growth of the areas served by the canals. In the broad picture of transportation, water complements the land carriers. It should be recognized that each form of transportation is best suited to their specific jobs. Certain bulk cargoes are best suited for water transportation, on the other hand, finished products can be handled better by land transportation means. Such use should be encouraged. To have an effective free enterprise system, all forms of transportation should be present so that agriculture, industry, and our citizenry will benefit to the utmost.

Future Trends in Waterways

Presently, the State of New York is at the crossroads of greater inland water transportation improvement and development. With the completion of the St. Lawrence Seaway, great interest has been re-aroused for the enlargement of the Champlain Canal and connecting this Canal with a like waterway to the St. Lawrence River at Sorel or Montreal. It is probable that such a waterway will develop the natural resources of northeast Canada to their full potential and be a boon to the economy of that area and to the Lake Champlain and Hudson River Basins as well as the eastern seaboard.

What the effect of the St. Lawrence Seaway will be on the Barge Canal is presently a conjecture. However, it is anticipated that the New York State ports, located on the Great Lakes, inland on the canals, and along the St. Lawrence River, will expand and develop. A logical outcome for those areas, therefore, should be expanded and new industry and increased population. Such growth will require a greater flow of raw and finished products and materials, into and out of these areas, many of which can be most economically moved by water. It would seem, therefore, that these two waterways will complement each other and will no doubt generate water-borne traffic for each other.

It appears from this paper that the destiny of New York, the Empire State, is tied in with canals and water transportation. The early canals brought pre-eminence to the State in agriculture, business, commerce and industry. They developed the interior of the Nation and made New York City the leading port and financial and manufacturing center in the Nation. There is much undeniable historical fact that the Barge Canal has helped the State and its cities to maintain this agricultural, financial and manufacturing pre-eminence. What benefits will accrue to the State and its people by the St. Lawrence Seaway and the development of deep water transportation from the Hudson to the St. Lawrence River by way of Lake Champlain, only time will tell. However, if the past and present benefits of canals can be taken as an indication of the future, then the continued growth of the economy of the State and a great portion of our Nation is assured.

Surely, no one can doubt that the people of New York State made a sound investment in building their canals.

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THE SUPPLY AND LOSS OF SAND TO THE COAST

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ABSTRACT

A summary of the various sources of supply and loss of sand to the coast is presented with special application to a reach of the coast of California. Where possible, a quantitative estimate of the annual amounts of sand supplied or lost is presented. The need is indicated for further investigation or research on those phases of the problem where methods and procedures are limited or totally lacking for estimating annual rates of supply or loss of sand.

INTRODUCTION

Man-made modification of the physiographical balance of the watersheds of the country has become of increasing importance as the population and the use of the land have increased over the years. Perhaps the first comprehensive discussion of this problem was that of Sonderegger (1935) where attention was called to the effect on the physiographical balance of such conservation measures as (a) changes in the watershed cover, (b) effect of regulation on the natural balance of a stream system, and (c) effect of debris barriers on the stability of the stream bed or the debris cone. Related to these problems is the following question which is often asked in coastal communities: "what is the effect of the construction of dams and other stream regulatory works on the supply of sand to the coast, with particular reference to the permanency and stability of our beaches?" To answer such a question requires an appraisal of the approximate magnitude of the various sources of supply and losses of sand to a given coast line. The most probable sources and losses of material are generally considered to be as follows:

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Sources of Sand Supply

(a) Major streams, (b) small streams and gullies, (c) cliff erosion and slides, (d) onshore movement of sand by wave action, (e) wind action.

Sand Losses

(a) Movement offshore into deep water, (b) losses into submarine canyons, (c) accretion against littoral barriers, (d) removal of sand for construction purposes, (e) wind action, (f) abrasion by wave action.

All of these factors may be important on some reaches of a shoreline; whereas, in other reaches only some of the factors may be effective, with others being either non-existent or of such minor importance that they can be neglected in an overall material balance. Methods are available to estimate the approximate magnitude of the quantity of some of the supply and loss factors, but in other cases the magnitude can be estimated only qualitatively. It is the purpose of this paper to review briefly the present state of knowledge on these factors and indicate where further investigations or research are necessary for a better evaluation of the processes involved.

In a study of a particular shoreline it is convenient to study each physiographic unit separately. The unit in this instance is defined as a shore area so limited that the shore phenomena within the area are not affected by the physical conditions in adjacent areas (Mason, 1950); that is, the energy and material available within the area are not dependent on adjacent areas. In some instances the boundaries of a unit are well defined; whereas, in other cases what is now considered to be a definite unit might after further study prove not to be such. Generally, the boundaries of physiographic units consist of such features as prominent headlands, man-made littoral barriers, submarine canyons, or other shoreline features, which prevent the movement of sediment into and out of the shore area under consideration.

The relative stability of a shoreline within a given physiographic unit is dependent on the material and energy available to the shore. Wave action is the primary source of energy, but since the wave characteristics are changing continuously, a particular shoreline apparently never reaches complete stability when short periods of time, such as days or weeks, are considered. However, from the long-time point of view, such as a year or a decade, where the supply and loss of material to the physiographic unit and the supply of wave energy are not altered by man-made structures, the shoreline area is comparatively stable. Thus, the average annual rate of supply of material is equal to the rate of loss for the average annual rate of expenditure of wave energy. Any man-made changes to any of these rates might well result in a progressive change in the shoreline configuration until a new condition is reached which will be in equilibrium with the altered material-energy balance. The time required to attain this new equilibrium condition depends to a great extent upon the relative magnitude of the various methods by which material is either supplied or lost from the shore area under study.

To illustrate the methods involved in making an appraisal of the supply and loss of sediment to a coastline the reach of the California coast extending from Point Lobos, south of Carmel, to Santa Barbara has been selected (Fig. 1). This reach has been selected because it appears to be isolated from the remainder of the coast by littoral barriers. That is, the Monterey Peninsula and the Carmel Submarine Canyon appear to be definite barriers by which no sand from the north coast can pass. Since the construction of the Santa Barbara breakwater in 1929, the harbor has served as a complete littoral

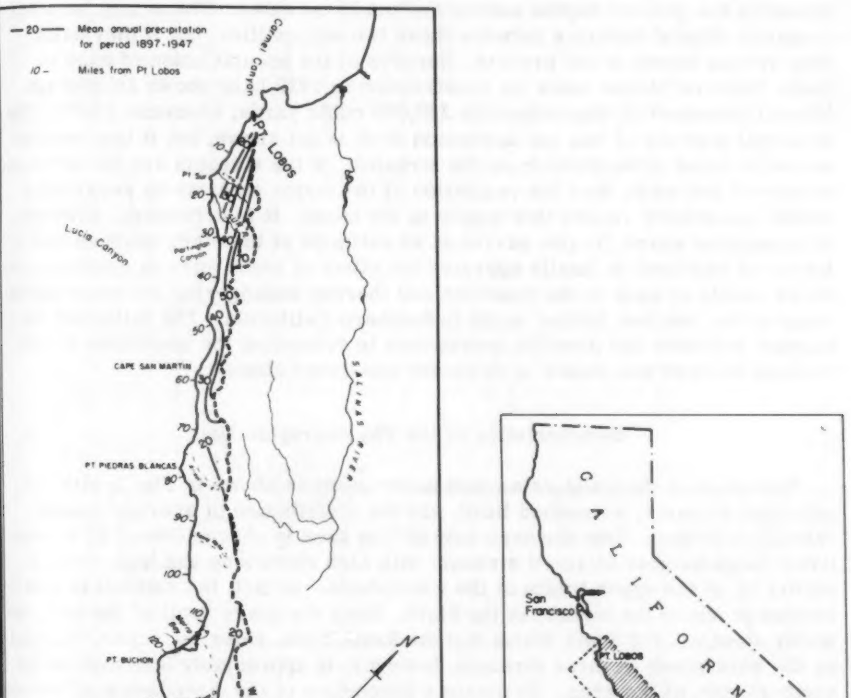

Fig. 1. Watershed map of reach of California coastline from Pt. Lobos to Santa Barbara.

barrier—except possibly for some of the relatively fine material which is moved in the greater depths seaward of the breakwater. There may be other complete littoral barriers between these two extremities, but if they exist, they are not known at the present. Surveys of the accumulation of sand in Santa Barbara harbor since its construction in 1929 have shown an average littoral transport of approximately 280,000 cubic yards, (Johnson, 1957). The principal sources of this net downcoast drift is not known, but it has been assumed to come principally from the streams. If the streams are the principal source of sediment, then the regulation of the major streams by reservoirs would appreciably reduce this supply to the coast. It is necessary, however, as mentioned above, to also arrive at an estimate of the other sources and losses of sediment to finally appraise the effect of reservoirs in causing a reduced supply of sand to the coastline and thereby endangering the recreational value of the beaches further south in Southern California. The following discussion indicates the possible approaches in evaluating the quantities of the various sources and losses of sediment mentioned above.

Characteristics of the Physiographic Unit

The reach of the California coast under study is shown in Fig. 1 with the principal streams, watershed limit, and the distribution of average annual rainfall indicated. The northern part of this area is characterized by a relatively large number of small streams with high elevations and high rainfall occurring at the upper limits of the watersheds. In fact, the rainfall in this section is one of the highest in the State. Near the lower limit of the unit, two major streams, the Santa Maria and the Santa Ynez, enter the ocean. Rainfall on the watersheds of these streams, however, is appreciably less than in the northern part of the area. To obtain a conception of the distribution of stream size, maximum elevation, and precipitation with distance along the coast, Fig. 2 has been prepared. For convenience, the groups of streams used by the California Department of Water Resources (1957) in planning the full utilization of California's water resources are indicated on this chart. One curve on Fig. 2 shows the accumulated drainage area in square miles starting with Point Lobos at the extreme northern limit of the unit and extending to the extreme southern limit at the Santa Barbara breakwater. It is evident from an examination of this curve that the contribution of drainage area in the Monterey-Carmel and the San Luis Obispo Groups, which involves about 148 of the 245 mile coastline, consists of relatively small streams. The major contributions of drainage area in the entire unit comes from the Santa Maria Valley and Santa Barbara Groups. The drainage area contribution from each group of streams and the average contribution per mile of coastline are summarized in Table 1. Of value in giving information on the number of drainage basins of various sizes, are the data shown in Table 2 as compiled from U.S.G.S. quadrangle sheets. The first line in the table shows the number of drainage basins in each size range. The second line shows the total square miles of drainage area in each size range, with the third line showing the percentage of the total drainage area in each range of size. It is of interest to note by inspection of lines 1 and 4 that 74 per cent of the entire drainage area is contributed by only 8 streams which have drainage areas of 50 square miles or greater.

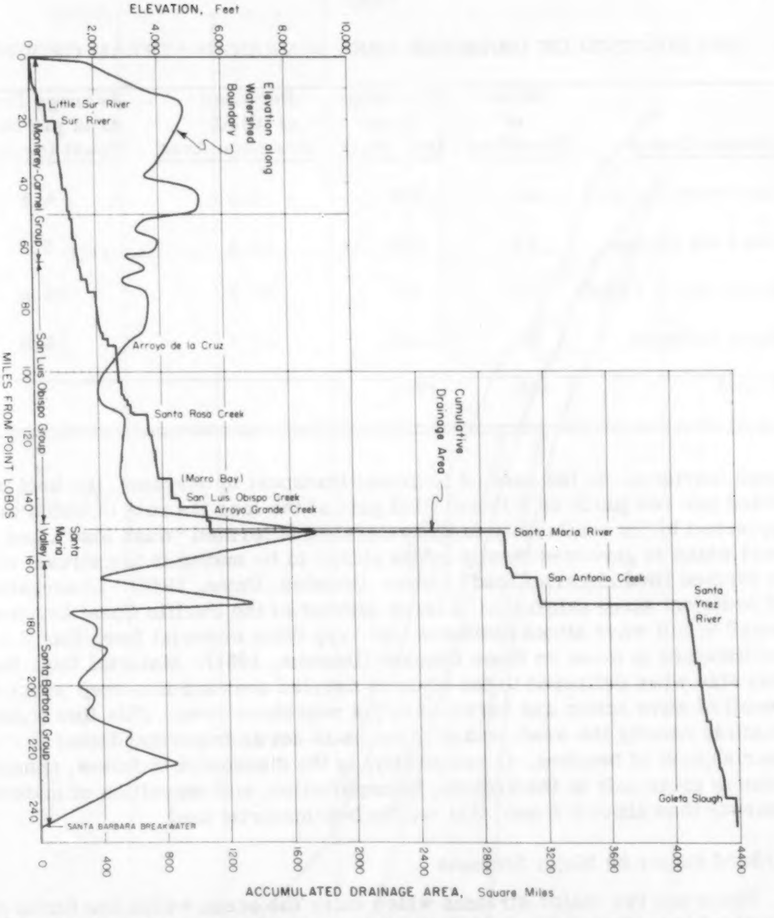

Fig. 2. Cumulative distribution of drainage area with distance from Pt. Lobos. Maximum elevation along the watershed boundary for the major streams also is shown.

The second curve in Fig. 3 shows the maximum elevation in the various drainage areas of the more important streams in the unit. This line represents the variation of elevation along the watershed limit of the entire unit shown in Fig. 1. The variation of average annual rainfall over the entire area is shown in Fig. 1. Thus, as stated above, the physiographic unit consists of small steep streams with high rainfall in the northern section with larger streams with less steepness and rainfall in the southern section.

Sources of Sand Supply

The sediment that is carried into the ocean from the land varies in size from the extremely fine material in the clay sizes to relatively large sand and

Table 1

DISTRIBUTION OF DRAINAGE AREA IN VARIOUS STREAM GROUPS

Stream Group	Miles of Coastline	Drainage Area (sq. mi.)	Per Cent of Total Drainage Area	Average Drainage Area per Mile of Coast (sq.mi/mi)
Monterey-Carmel	65	290	6.6	4.5
San Luis Obispo	83	680	15.5	8.2
Santa Maria Valley	12	1980	45.2	165.0
Santa Barbara	85	1430	32.7	16.8
Totals	245	4380		

rock fractions. In the case of sediment transport by streams, the load is divided into two parts as follows: that part of the load the rate of which is governed by its availability in the watershed is termed "wash load", and that part which is governed mostly by its ability to be moved in the stream channel is termed "bed material load" (Amer. Geophys. Union, 1947). Observations of sediment sizes existing on a large number of the Pacific Coast beaches exposed to full wave attack indicates that very little material finer than 0.2 millimeters is found on these beaches (Bascom, 1951). Material finer than this size when delivered to the coast is carried seaward into deep water as a result of wave action and currents in the nearshore area. This fine material which is usually the wash load of streams is not an important factor in the nourishment of beaches. Consequently, in the discussion to follow, consideration is given only to the erosion, transportation, and deposition of material coarser than about 0.2 mm, that is, the bed-material load.

(a) Sand Supply by Major Streams

There are two major streams which enter the ocean within the limits of the length of shoreline under study. These are the Santa Maria River (1843 sq. mi. drainage area) and the Santa Ynez River (924 sq. mi. drainage area). The

Table 2

NUMBER AND SIZE OF INDIVIDUAL DRAINAGE BASINS

	Size of Individual Drainage Basins Sq. miles													
	Less than 1	1 - 2	2 - 3	3 - 4	4 - 6	6 - 8	8 - 10	10 - 15	20 - 30	30 - 40	40 - 50	50 - 75	75 - 100	More than 100
Number of drainage areas in each size range	186	64	18	21	13	9	9	7	5	2	3	2	2	4
Square miles of area in each size range	79	89	45	71	63	60	80	87	86	74	135	114	165	3087
Per cent of total area in each size range	1.8	2.0	1.1	1.6	1.4	1.4	1.8	2.1	2.0	1.7	3.1	2.6	3.8	70.2
Percentage of total area smaller than given size range	1.8	3.8	4.9	6.5	7.9	9.3	11.1	13.2	15.2	20.3	23.4	26	29.8	100

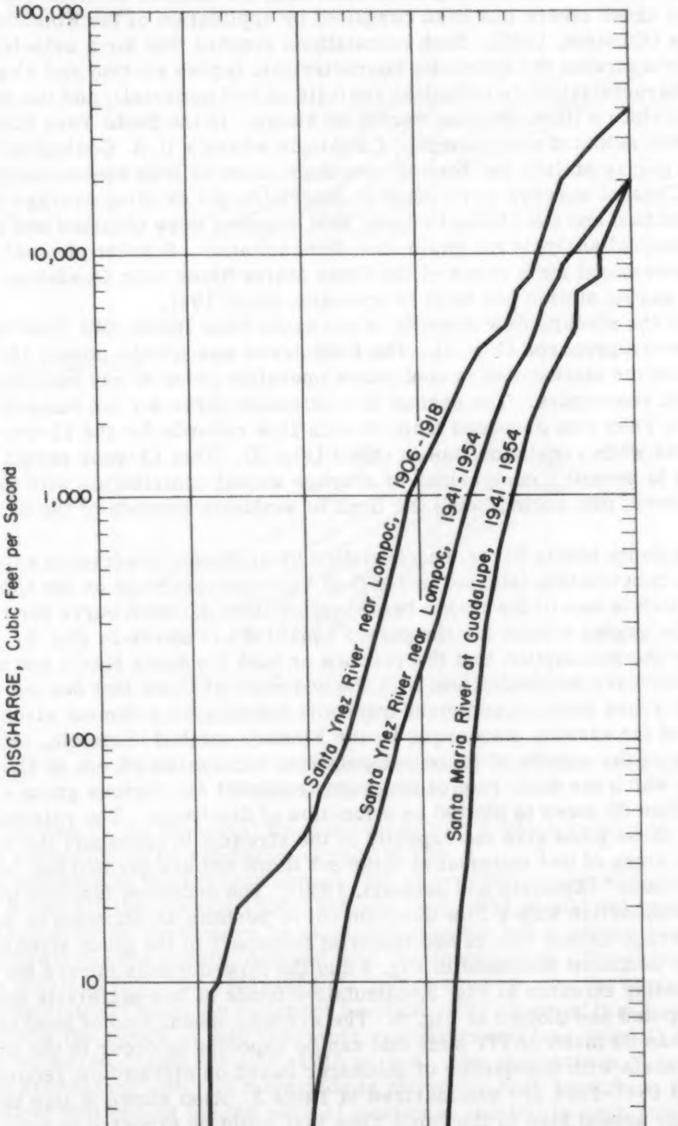

Fig. 3. Flow duration curves for the Santa Ynez and Santa Maria Rivers.

average annual rate of transport of sand coarser than an 80 mesh sieve (0.177 mm) for these rivers has been computed by application of the Einstein bedload formula (Einstein, 1950). Such calculations require that for a selected alluvial reach of a stream the hydraulic characteristic (cross section and slope), sediment characteristics (mechanical analysis of bed material), and the discharge characteristics (flow duration curve) be known. In the Santa Ynez River a reach was selected near Lompoc, California where a U. S. Geological Survey stream gaging station has been in operation more or less continuously since 1906. Channel surveys were made to determine the existing average channel cross section and the channel slope. Bed samples were obtained and subjected to mechanical analysis for grain size determination. Similar channel data were determined for a reach of the Santa Maria River near Guadalupe where a stream gaging station has been in operation since 1941.

From the stream-flow records of the Santa Ynez River, two flow-duration curves were prepared (Fig. 3). The first curve was for the period 1906 to 1918 when the station was in continuous operation prior to any regulation by upstream reservoirs. The second flow-duration curve for the same station on the Santa Ynez was prepared from stream flow records for the 13-year period 1941-1954 when regulation was in effect (Fig. 3). This 13-year period was selected to permit a comparison of average annual contribution with the Santa Maria River, this period being the limit of available records of the Santa Maria.

In the Santa Maria River, no regulation by upstream reservoirs existed until the construction (started in 1957) of Vaquero reservoir on the Cuyama River which is one of the major branches. A flow-duration curve for the Guadalupe gaging station for the period 1941-1954 is shown in Fig. 3.

Under the assumption that the reaches on both the Santa Maria and the Santa Ynez Rivers are in equilibrium with the sequence of flows that has occurred in recent years rates of sediment transport for various sediment sizes was calculated for various discharges by the Einstein method (Einstein, 1950). A summary of the results of these computations for the two rivers is shown in Fig. 4 in which the daily rate of sediment transport for various grain sizes greater than 80 mesh is plotted as a function of discharge. The relationships shown in these plots give the capacity of the streams to transport the various sediment sizes of bed material at different flows and are termed the "sediment functions" (Einstein and Johnson, 1950). The sediment function when used in conjunction with a flow duration curve permits an estimate to be made of the average annual rate of bed material transport in the given stream. By use of the sediment functions in Fig. 4 and the flow-duration curves for the corresponding streams in Fig. 3, cumulative totals of bed-materials load have been computed and plotted in Fig. 5. The average annual load of sand of sizes greater than 80 mesh (0.177 mm) that can be expected to occur in the present river channels with a sequence of discharge based on streamflow records for the period 1941-1954 are summarized in Table 3. Also shown in this table is the average annual load in the Santa Ynez that could be expected to occur in the present river channel if an average sequence of flows should occur as observed in the period 1906-1918 and characterized by the flow duration curve for this period (Fig. 3). Examination of the first two lines in this table shows that the annual load in the Santa Ynez River was approximately 3 times that of the Santa Maria River for the flow conditions based on the same period of record (namely 1941-1954). This difference in annual load of the two rivers can undoubtedly be attributed to the difference in the average annual rainfall

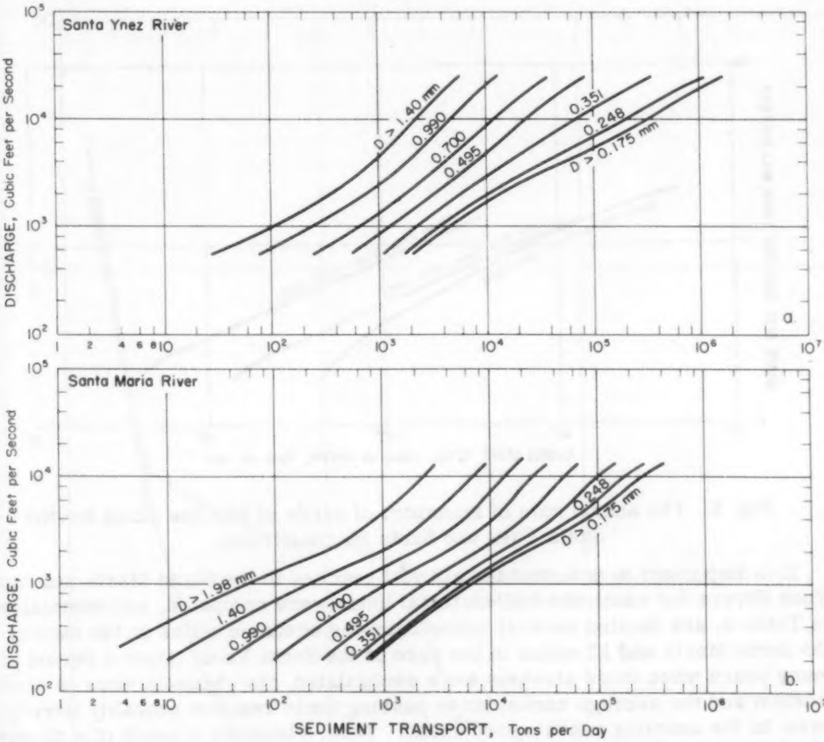

Fig. 4. The Einstein bed-load function for the Santa Ynez and Santa Maria Rivers.

as evidenced by the isoheytal lines shown in Fig. 1 which in turn is reflected in the character of flow duration curves in Fig. 3.

Comparison of the average annual rate of bed-material load in the Santa Ynez River for the periods 1906-1918 and 1941-1954 should not be interpreted that regulation by reservoirs as existed during the latter period was responsible entirely for the greatly reduced load in recent years. The flow-duration curve obtained from the 1941-1954 discharge records was the basis for annual load calculations from the sediment function. The shape of this flow-duration curve was the result of both reservoir regulation and less rainfall than that which occurred during the 1906-1918 period. The comparison of annual rainfall in decreasing order of magnitude is shown for both periods in Fig. 6. It is evident that except for the year of maximum rainfall in each period the annual rainfall in the 1905-1918 period was invariably the highest of the two periods. Although these rainfall records are from Santa Barbara, which is outside of the Santa Ynez drainage basin, the rainfall in the two areas probably were closely related, such that the data in Fig. 6 indicates the relative amounts of rain which occurred in the Santa Ynez basin in the 1906-1918 and 1941-1956 periods.

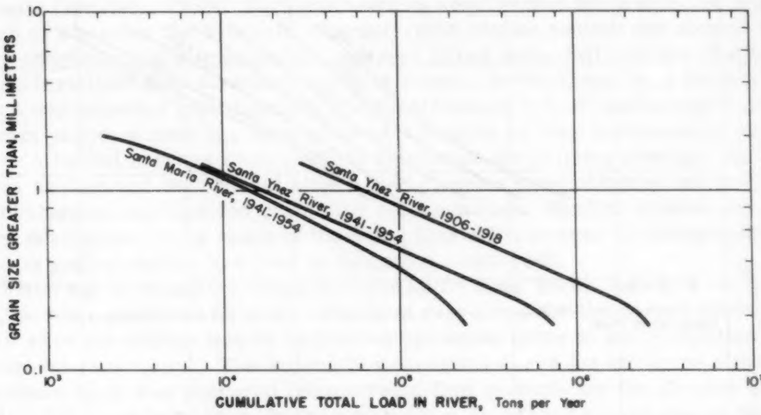

Fig. 5. The annual rate of transport of sands of various sizes for the Santa Ynez and Santa Maria Rivers.

It is important to note that the alluvial reaches in the Santa Maria and Santa Ynez Rivers for which the bed-material loads were computed, and summarized in Table 3, are located several miles from the ocean—6 miles in the case of the Santa Maria and 12 miles in the case of the Santa Ynez. Over a period of many years when these streams were unregulated, the channels were in equilibrium and the average annual loads passing these reaches probably were equal to the amounts reaching the ocean. Thus, whenever a reach of a channel which has a sediment function, and which is usually called an alluvial reach, shows a low or zero rate of deposition or scour, the sediment function may be interpreted over a given length of time to determine the total or average bed-sediment supply of the drainage basin above. Basically, the sediment supply is naturally the primary factor determining the behavior of the stream, and in the course of centuries the stream channel has been built up by sediment

Table 3

ANNUAL BED-MATERIAL LOAD OF MATERIAL COARSER THAN 80 MESH IN THE SANTA MARIA AND SANTA YNEZ RIVERS

River	Period of flow-duration data	Average annual load tons/year	cu.yds./year
Santa Maria	1941-1954	240,000	178,000
Santa Ynez	1941-1954	770,000	570,000
Santa Ynez	1906-1918	2,600,000	1,930,000

*Unit weight of sand assumed = 100 lbs/cu.ft.

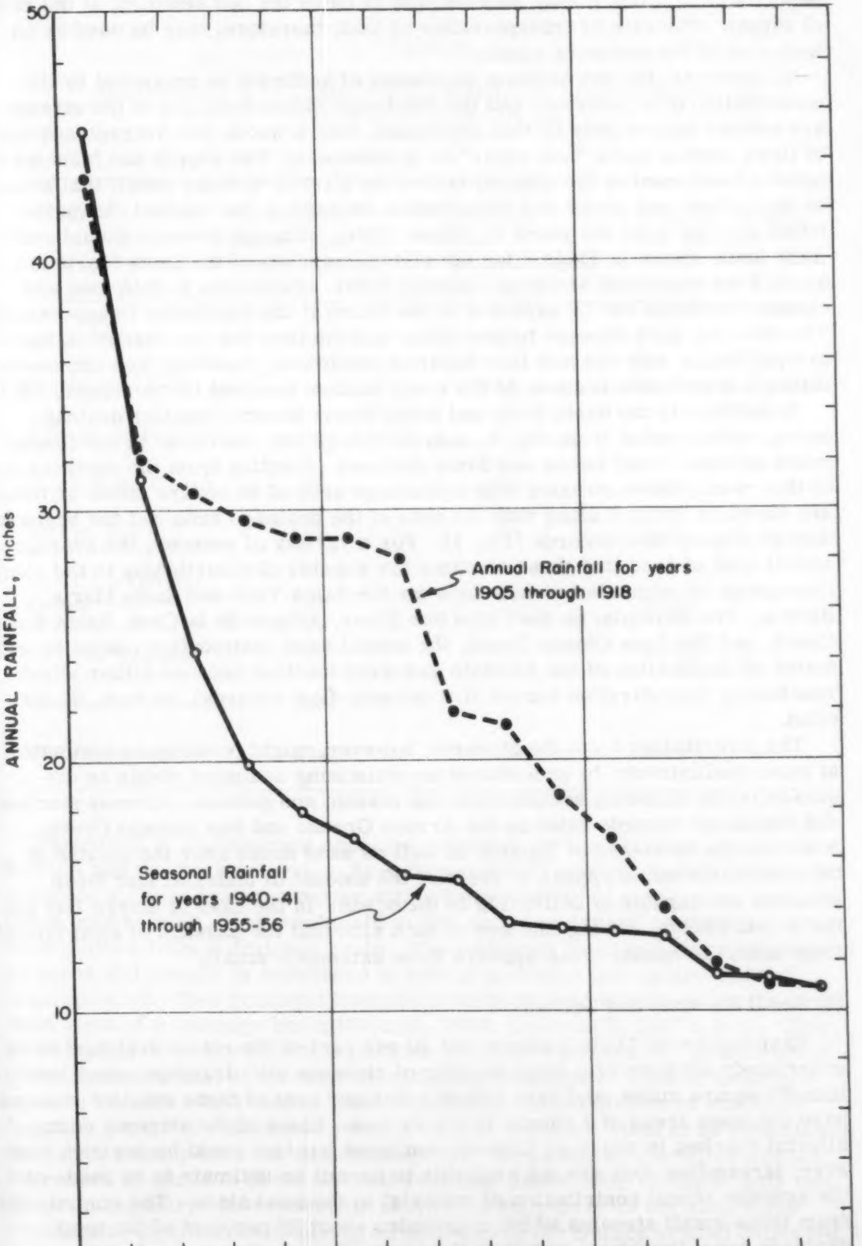

Fig. 6. Annual rainfall at Santa Barbara, California for the periods 1905-1918 and 1940-1956.

deposits until it has finally become able to move the bed sediment at the rate of supply. The rate of transportation of load, therefore, may be used as an indicator of the sediment supply.

If, however, the downstream movement of sediment is prevented by the construction of a reservoir and the discharge characteristics of the stream are altered appreciably by this regulation, then a whole new average sequence of flows occurs and a "new river" is in existence. The supply and transportation of sediment in the channel is thereby altered with the result that scour or deposition may occur and progressive changes in the channel characteristics develop over the years to follow. Thus, although average annual sediment loads shown in Table 3 for the alluvial reaches of the Santa Maria and Santa Ynez represent sediment capacity today, alterations in both load and channel condition can be expected in the future if the regulation is appreciable. The time for such changes to take place and the time for the channel to come to equilibrium with the new flow duration conditions, however, are extremely difficult to estimate because of the many factors involved (Sonderegger, 1935).

In addition to the Santa Ynez and Santa Maria Rivers, smaller drainage areas, as is evident from Fig. 1, also discharge into the ocean in the coastal reach between Point Lobos and Santa Barbara. Starting from the northern part of this reach, those streams with a drainage area of 40 square miles or more are shown in Table 4 along with the size of the drainage area and the beginning date of stream flow records (Fig. 1). For a variety of reasons, the average annual load of sand that these streams are capable of contributing to the shoreline cannot be estimated as was done for the Santa Ynez and Santa Maria Rivers. For example, on the Little Sur River, Arroyo de la Cruz, Santa Rosa Creek, and San Luis Obispo Creek, the annual sand contribution cannot be estimated by application of the Einstein sediment function because either alluvial reaches or flow-duration curves (i.e. stream-flow records), or both, do not exist.

The contribution from the streams, however, might possibly be estimated, at least qualitatively, by procedures on estimating sediment yields as discussed in the following section on small stream and gullies. Alluvial reaches and discharge records exist on the Arroyo Grande and San Antonio Creek; however, the existence of lagoons as well as sand dunes near the mouths of these streams would appear to restrict the amount of material that these streams are capable of delivering to the ocean. In the case of Morro Bay and the Goleta Slough, the lagoons are of such size that the passage of sand through them from the upland areas appears to be extremely small.

(b) Small Streams and Gullies

Examination of Table 2 shows that 20 per cent of the entire drainage basin under study consists of a large number of streams with drainage areas less than 40 square miles, and over seventy-five per cent of these smaller streams have drainage areas of 2 square miles or less. Some of the streams contain alluvial reaches in which an Einstein sediment function could be derived; however, streamflow data are not available to permit an estimate to be made of the average annual contribution of material in the sand sizes. The contribution from these small streams which constitutes about 20 per cent of the total drainage area, therefore, requires the application of other procedures in predicting sediment yields.

Table 4

SECONDARY STREAMS WITH DRAINAGE AREAS IN EXCESS OF 40 SQUARE MILES

Stream	Distance of Mouth from Pt. Lobos (miles)	Drainage Area (Square miles)	Period of Stream Flow Record
Little Sur	15	41	None
Sur River	20	61	Since 1950
Arroyo de la Cruz	75	45	Since 1950
Santa Rosa Creek	92	49	None
Morro Bay	113	80	None
San Luis Obispo Creek	134	85	None
Arroyo Grande	143	169	Since 1939
San Antonio Creek	166	151	Since 1941
Goleta Slough	236	53	Incomplete

Various U. S. Government agencies, particularly the Soil Conservation Service and the Forest Service, have been actively engaged in determining the sediment yield in drainage basins with a large variety of size, topography, cover, and rainfall. Sediment yield in this sense is defined as the total sediment outflow from a drainage basin. The sediment yield includes sediments of all sizes and usually is expressed in tons of sediment per square mile of drainage area. Two principal methods have been used in estimating the sediment yield of a drainage basin (Glymph, 1954; Gottschalk, 1957 a & b). They are: (a) Surveys of sediment deposition in reservoirs, and (b) Measurement of the sediment load of streams by sampling techniques. Both of these procedures require observations extending over a period of time of sufficient length to permit a reasonable average rate of sediment yield to be established. Such data usually have the disadvantage that the sediment quantities are not expressed in amounts of the various sediment sizes. For example, only that material measured in a reservoir sedimentation survey of the sand sizes would be of significance in giving an estimate of the quality of sand delivered to the coast line; however, only total amounts of sediment usually are given.

In the case of suspended sediment load measurements, only the fine material in suspension usually is sampled, with the result that the quantity of sand carried by the stream is underestimated by this procedure. Unfortunately

for the area under consideration in this study little or no data using the above two procedures have been obtained. The extent of such data has been summarized by Flaxman and High (1955). The observations show that only reconnaissance type information is available in limited areas is the drainage basins of the Santa Maria and Santa Ynez Rivers.

Considerable study has been made by the U. S. Department of Agriculture with a view to predicting sediment yield on the basis of erosion measurements in a drainage area. Such studies involve extensive plot studies as well as estimates of sediment delivery rates by sheet and channel erosion (Glymph, 1957). Here again, however, as in the case of the direct measurement of sediment yield by either sedimentation surveys or sediment-load sampling, little or no data have been obtained for the coastal area under discussion (Flaxman and High, 1955). Sediment yield predictions by the erosion method has the same disadvantage as the other methods—namely, that the portion of the sediment yield which is in the sand sizes is not given.

Where basic data become available on any of the above procedures for estimating annual sediment yield (refined to give the yield in the sand sizes), estimates of annual sand contribution by small streams to the coastline under study perhaps can be made by transposition of data from comparative drainage basins. For example, a procedure in estimating the sediment yield of streams with similar drainage area characteristics has been developed by Anderson (1949 and 1957). This method relates annual sediment accumulation to such factors as maximum yearly peak discharge, area of main channel of the drainage basin, and cover density on the drainage basin. The acquisition of the necessary field data for application of the method to the coastal area under consideration is beyond the scope of the present study; however, a program to obtain the required basic data could be established. A necessary requirement in the basic data would be the determination of yield of the sand sizes rather than merely the total sediment yield.

(c) Cliff Erosion and Slides

The contribution of material from cliffs by direct wave action probably is relatively small for most of the coastline under consideration. The rugged shoreline from Point Lobos to about Morro Bay consists of a rock very resistant to erosion. Also, these rocks consist of material which produce sediments which are mostly finer than sand when erosion does occur. Southward of Morro Bay it appears that erosion of some of the cliff areas might contribute an appreciable supply of sand to the shoreline. This appears to be the case in particular on the leeward (southern) side of prominent headlands where erosion of the less resistant base material has progressed and formed hook bays which face southward. For example, the low cliffs south of Point Purisima consist of a rather loosely consolidated sandstone which at extreme high tide are subjected to direct wave attack. This eroded material contains a relatively large quantity of sediment in the sand sizes. Northward of the prominent headlands cliff erosion by direct wave attack usually is less pronounced because an accumulation of sand against the headland exists to form a wide beach and thereby afford protection of the cliff against wave action.

To arrive at even an approximation of the annual contribution of sand to the shoreline by cliff erosion would require rather detailed investigations which are beyond the scope of this study. In addition to an examination of old maps and photographs certain detailed and systematic field studies would be

necessary, such as was done by Chieruzzi and Baker (1958) in their studies of bluff recession along Lake Erie, to appraise the annual contribution of sand by cliff erosion.

Closely related to cliff erosion is the contribution of material by slides induced either by natural forces or man made operations such as road cuts, etc. In some localities the cliffs are overlain with an easily eroded material which might be moved into the ocean by such factors as failure of the underlying cliff by wave action, rain impact, sheet and concentrated runoff, weathering action, frost action, subsurface moisture, etc. (Chieruzzi and Baker, 1958). To arrive at an estimate of the annual sand contribution requires rather detailed field studies as well as information from local inhabitants, as to the extent of slides which have occurred in the past. For example, the California Division of Highways has certain valuable information on slide material wasted into the ocean along State Highway Route 1 between Point Lobos and Morro Bay. The most critical reach for such contributions is located in the 62 mile reach between Garrapata Creek on the north and San Carpojo Creek to the south (8.5 and 70.5 miles, respectively, below Point Lobos). A summary of the various localities and amounts of sediment as presented by the California Division of Highways (1957) is as follows:

1.	Material disposed of which results from gutter slough and minor small slides during a normal one-year period	5,000 cu. yds.
2.	Annual roadbed restoration at Willow Creek Slipout, 65 miles south of Monterey	500 cu. yds.
3.	Annual roadbed restoration at White Creek Slipout, 69 miles south of Monterey	500 cu. yds.
4.	Slide material washed into ocean at Little Redwood Canyon, 40 miles south of Monterey during December 1955 storm	30,000 cu. yds.
5.	Slide material deposited on slope to ocean near Limekiln Creek in August 1952 and since December 1955 storms	200,000 cu. yds.

The above estimate of quantities is based on current slide activity in the area and would not be indicative of the quantities of slide material that may have reached the ocean in the earlier period following the original construction of this road when the cut slopes were far less stable than at the present time.

Items 2 and 3 cover areas where there is a continual and gradual vertical settlement of the roadbed section which requires about 500 cubic yards of material a year at each location to maintain the grade across the areas. It is to be recognized that the material contributed by these slides is a gross amount, and the quantity of material of the sand sizes would be substantially less than those given.

Other locations where slides have occurred from time to time are southward from Point Arguello. The best known slide areas are those localities where the Southern Pacific Railroad tracks have been damaged. The amounts of material contributed by such slides, however, is not known.

(d) Onshore Movement of Sand by Wave Action

The rate of movement of sand in fairly deep water as induced by wave action is impossible to evaluate with the present state of knowledge of this process; however, investigations of the fundamental mechanics of this process

by Li (1954), Manohar (1955), and Kalkanis (1957) should eventually provide a basis for evaluating the contribution of sand to a shoreline by this mode of transport. A study of offshore sediments in the vicinity of Southern California promontories by Trask (1955) indicates that sand probably is moved by wave action at relatively large depths. This finding was confirmed by the work of Inman (1957) where underwater photographs showed sand ripples caused by wave action to exist at great depths.

General evidence indicates that the annual rate of movement onshore from offshore areas perhaps is small. This conclusion is based on three tests by the Corps of Engineers in which dredge material was spoiled relatively close inshore with the expectation that wave action would transport the sand onshore to replenish that lost by erosion. These tests were made at Long Branch, N. J., Atlantic City, N. J., and Santa Barbara, California (Hall and Herron, 1950; Hall, 1952; Harris, 1954). The general conclusion from these tests was that "From present evidence—this method will not provide nourishment at a suitable rate to justify its general use" (Hall, 1952).

(e) Wind Action

In the reach of the coast from Point Lobos to Santa Barbara there appears to be no contribution to the coast by wind action because the prevailing winds are onshore. In those instances where a low area exists landward of a prominent headland, such as Point Sur, Piedras Blancas, Point Purisima, etc., sand is moved by wind action from the area of accumulation on the northern side of the point and re-enters the ocean on the southern or leeward side. The net result of this action does not provide a supply of sand to the coast.

Sand Losses

All of the sand that is delivered to the shoreline of the physiographic unit under study does not eventually reach the harbor at Santa Barbara because of losses which occur as a result of certain natural forces. The more important causes of sand loss to a shoreline were listed above in the Introduction. A brief discussion of these various items as they relate to the Point Lobos - Santa Barbara reach of coastline is as follows:

(a) Movement Offshore into Deep Water

Material delivered to a shoreline by any of the methods discussed above under the heading "Sources of sand supply" might be moved into water offshore and come to rest in depths too great for natural forces to again move it back onto the shore. There appear to be several phenomena which could create such a condition. First, the force of the supplying agency, whether it is by the current of a river or by wind action, might initially carry the sediment into depths which are too great for the forces of wave action and coastal currents to later move the material shoreward. Also, material delivered to the surf zone is sorted by wave action, with the finer fractions being carried seaward, with only the coarser fractions being able to remain in the very turbulent surf zone. Bottom samples generally show a reduction in mean grain size with distance seaward from the shoreline. In times of exceedingly high wave action a shoreward movement of material from offshore probably occurs as a result of the forces discussed by Li (1954), Manohar (1955), and

Kalkanis (1957); however, the normal condition is for a sorting action to be taking place in and near the surf zone.

(b) Losses into Submarine Canyons

Where material is moved along a shoreline as littoral drift, a loss of sand may occur where the head of a submarine canyon comes close into shore. In such a case the sand falls into the canyon head and continues to accumulate. Periodically, the fill becomes unstable and slides down the canyon beyond the reach of forces which tend to move the material onshore (Shepard, 1951). This sand in the canyon therefore becomes permanently lost as a source of sand supply to the downcoast shoreline. The submarine canyons along the Pacific coast were first studied by Davidson (1897). In more recent years, Shepard and Emery (1941) discussed the canyons along the California coast. As previously mentioned, the Carmel Canyon is assumed to serve as a natural littoral barrier at the northern end of the reach of the coast under study. Three canyons exist between Point Lobos and the breakwater at Santa Barbara which is the southern limit of the reach. These are the Sur-Partington, Lucia, and Arguello canyons, (Fig. 1). The extent to which these canyons serve as littoral barriers is not known, but the work of Shepard and Emery (1941) indicates that perhaps the Partington canyon is the only barrier because the head of this canyon extends almost to the beach; whereas the other canyons head at a considerable distance from shore. The Sur and Lucia canyon head about 3 miles from land, and the Arguello Canyon heads 10 miles off the coast.

The only known measurement of sand movement into submarine canyons is that at the La Jolla Canyon near the Scripps Institution of Oceanography. This submarine canyon probably is not representative of all the canyons, and also the measurements to date are too meager to permit even an approximate estimate of the average annual loss of sand by this process. Of considerable importance to the supply of sand to Southern California beaches would be studies of the sand loss into the Hueneme and at port canyons Point Dume. These canyons are, of course, beyond the limits of the present study; but they are of importance to a comprehensive study of the entire California shoreline.

(c) Accretion against Littoral Barriers

Littoral barriers may be natural or man-made, and they may be partial or complete. A natural barrier might be either a headland or a submarine canyon. In some instance, headlands which previously were considered to be complete littoral barriers now appear not to be barriers. For example, Trask (1955) in studies of bottom conditions in the vicinity of Point Conception, Point Arguello, and Point Dume showed that sand definitely moves around these Southern promontories. This movement probably takes place in the larger depths offshore by the mechanism discussed by Li (1954), Manohar (1955) and Kalkanis (1957), as well as along the base of the promontories themselves by turbulence as suggested by Chien (1956). The accumulation on the upcoast side of such headlands has proceeded throughout geological time, such that an equilibrium condition in terms of sediment supply, upcoast accretion, general shoreline configuration, etc. appears to have been established. That is, on a long-term average the sediment supply reaching the upcoast side of the headland is equal to that which moves around the promontory and on downcoast. On a short-term basis, however, such a natural barrier may serve as a partial barrier. The natural forces which deliver the littoral material to the upcoast

side of a headland may be insufficient to move the material around the barrier. Consequently, accretion will occur in the upcoast area until the temporary storage volume is exhausted or wave action and other natural forces become sufficiently large to move this temporary accumulation of material out of the storage area. Geologically speaking, therefore, the accretion at a headland may reach a condition where the headland no longer serves as a complete littoral barrier; however, on the short-term basis of a single season or a prolonged climatological cycle, an accumulation of material on the upcoast side of a promontory may occur and result in an erosion problem on the downcoast side.

A submarine canyon, in some instance, may serve as both a complete and partial littoral barrier. The accumulation at the head of a canyon may become of such a size that littoral material bypasses the canyon. A slide may then occur and the canyon becomes a complete barrier until the accretion reaches such proportions that littoral material again passes the canyon head.

In the case of man-made littoral barriers, shoreline structures often act as complete littoral barriers—at least for a period of time after completion of construction. This period of time, of course, depends on the distance that the structure projects into the littoral zone. Following construction of a jetty or breakwater accretion occurs on the upcoast side and erosion usually occurs downcoast. Accretion continues until the littoral compartment is filled. Material then starts to move around the barrier and tends to re-establish equilibrium conditions.

In the reach of shoreline under study three breakwaters have been constructed. These are located at Morro Bay, San Luis Harbor, and Santa Barbara. The Santa Barbara breakwater at the southern end of the reach appears to serve as a complete littoral barrier. Measurement of the accretion at Santa Barbara has permitted a reliable estimate of the rate of littoral transport to be made for this section of the coast (Johnson, 1953).

The breakwaters at Morro Bay appear to have reached an equilibrium condition and littoral material apparently is moving past the harbor entrance. At San Luis Harbor the breakwater was built on a rocky underwater ridge. This structure apparently does not interrupt the littoral drift (if there is any) as there is no accumulation inside the breakwater tip as occurs at Santa Barbara. Because the San Luis Breakwater was constructed on a rocky underwater ridge, it is possible that, prior to construction of the breakwater as well as at the present, there is sufficient turbulence to insure that littoral materials completely bypass the harbor. There has been no maintenance dredging in the harbor, but recent surveys do not appear to be available to determine whether or not shoaling of the area is actually occurring.

(d) Removal of Sand for Construction Purposes

At a few localities along the California coast, sand and gravel have been removed from beaches and nearshore areas for construction and other commercial uses. Removal of such material from below the high tide line can be done only by State permit, and the quantities removed must be reported. In addition to the direct removal of material from the beaches, sand and gravel often is removed from stream beds upstream from the high tide line. Depending on the distance from the ocean this practice can be as effective as direct removal from the beaches in reducing the available supply of sand downcoast from the stream mouths; however, the removal of sand from beaches and

streams for the Point Lobos-Santa Barbara reach of shoreline has been relatively small. For example, data compiled by the California Division of Mines (1957) show the following quantities of sand removed from this reach for the 10-year period, 1945-1955, inclusive:

Type of Deposit	Location City	County	Amount removed (short tons)
Beach	Oceano	San Luis Obispo	37,000
Beach	San Simeon	San Luis Obispo)	
Beach	Canbria	San Luis Obispo)	5,700*
Santa Ynez River	Solvang) Buellton) Lompoc)	Santa Barbara	150,000
Santa Maria River	Sisquoc	Santa Barbara	583,000

*Only for 1954 and 1955

It should be recognized that the removals from the Santa Ynez and Santa Maria Rivers were made at relatively large distances from the ocean and any effect on the stability of beaches by the removal of such relatively small quantities would be difficult to estimate. Also much of the material removed from these two rivers is of the gravel and small boulder sizes which are of small importance as a source of sand to the coast.

(e) Wind Action

Several areas exist along the Point Lobos-Santa Barbara reach of shoreline where considerable amounts of sand apparently are moved inland from the beaches by wind action. These areas generally are on the upcoast side of headlands where sand has accumulated over geological time as discussed in item c. The prevailing onshore winds acting on these areas of deposition have moved sand inland, sometimes for several miles. Examination of aerial photographs of the coastline has shown that the areas of most active sand movement by wind are in the vicinity of Morro Bay and in the reach which extends from Pismo Beach southward almost to Point Arguello (Fig. 1). Minor sand losses occur in the vicinity of Point Sur, Point Sierra Nevada, Piedras Blancas Point, San Simion Point, east of Goleta Point, and upcoast from the Santa Barbara breakwater. In a discussion of the geology of the Santa Maria district, Woodring and Bramlette (1950) have considered the dunes of the area to consist of the following sands:

Modern dune sand; Actively drifting sand
Intermediate dune sand; Dune sand more or less anchored by vegetation
Old dune sand; Dune sand anchored by vegetation

The important areas of active sand movement (modern and intermediate dune sands) are indicated in Fig. 1. Not shown in this figure are the inactive or old dunes which extend many miles inland, particularly in the Santa Maria Valley. Old dunes are found elsewhere along the southern California coast, as for example, between the north border of the Palos Verdes Hills and Playa del Rey, in Los Angeles County (Woodring, Bramlette, and Kew, 1946). The reason

for the present inactivity of the large area of old dunes is unknown but probably is the result of a climatic change, greater precipitation or decreased wind velocity, or both.

Considerable study over the years, primarily by geologists, ecologists and geographers, has been devoted to sand dunes and related problems. Most of these studies have been concerned with dune forms as related to sand supply, vegetation, wind, precipitation and other factors (Melton, 1950; Hack, 1951). Some data are available, however, on the distance which dunes advance per year. For example, Ranwell (1958) summarizes the annual rates of dune movement as measured by various observers, as well as presenting certain measurements of his own in Wales. These data are presented in Table 5.

Table 5

RECORDED RATES OF DUNE MOVEMENT
(After Ranwell, 1958)

Location	Rate feet per year
Inland	
Indiana	3 - 6
Lake Michigan	6 -13
Coastal	
Kurische Nehrung, Germany	18-20
Gascony, France	30 (mean)
Morfa Harlech, Wales	12 (max.)
Morfa Duffryn, Wales	20 (max.)
Newborough Warreln, Wales	22 (max.)
Great Crosby, South Lancashire	3.6
Freshfield, South Lancashire	18-24
Norfolk Coast	5

Crude as these various measurements probably are, such data do permit a rough estimate to be made of the annual rate of sand that could be moved inland from a beach. For example, with a dune 50 feet high which moves about 10 feet per year, approximately 2000 cubic yards of sand per year would be transported per one hundred feet length of dune crest.

In addition to these rather qualitative studies on sand movement, considerable work has been done on the fundamental mechanics of sand transport by wind. The earliest work in this field was that of Bagnold (1954) which involved both laboratory and field studies. His studies showed that the rate of sand transport is a function of the shear stress and the sediment characteristics. Later, work in this field was done by Zingg (1953) and Cheffel (1945, 1946). Unfortunately, the procedures and techniques are not sufficiently well developed to permit the calculation of the annual rate of sand movement away from a

reach of shoreline such as that in the vicinity of the mouth of the Santa Maria River. Field studies with equipment for measuring rates of sand transport and the velocity distribution of wind appear to offer the best possibility of arriving at reliable estimates of annual rates of sand loss by wind in various localities. Such studies, however, were beyond the scope of the present investigation.

(f) Abrasion by Wave Action

One effect of the violent turbulence in the surf zone is to abrade the sediment supplied to that region. The relatively fine material resulting from such abrasion is carried seaward by currents and probably thereafter is never carried onshore to become an important part of the beach sands. Some studies on the problem of sand abrasion have been made (Mason, 1942) and apparently the loss of material from the littoral zone by this process is extremely small and, therefore, not an important factor in the overall material balance to a shoreline.

SUMMARY

In the reach of the California coastline between Point Lobos and Santa Barbara, approximately 280,000 cubic yards of sand per year are transported as littoral drift and deposited in Santa Barbara Harbor. This quantity of material is the net production of sand resulting from the various processes of supply and loss in this reach of shoreline. From a consideration of the various sources of supply, it is evident that the two major streams—the Santa Ynez and the Santa Maria Rivers—alone are capable of delivering a major portion of the material which eventually reaches the downcoast point of Santa Barbara. It appears that the remaining streams in the drainage area are the next most important contributor of material, followed by cliff erosion and slides; but even approximate quantities of material from these sources are difficult to estimate at present. Similarly the contribution of sand from offshore sources is completely an unknown factor, but it is probably relatively small. The contribution of sand to the coast by wind action is negligible since the winds generally are onshore. Since the streams appear to be the major contributors of sand to the coast, any factor or factors which tend to reduce this source of supply would eventually tend to affect the stability of the beaches in the area. It is possible, however, that the reduction in sand supply from one source might influence the other natural processes of supply, for example —cliff erosion, such that downcoast delivery of sand by littoral drift will not be greatly reduced in quantity.

Of the total amount of material delivered to the littoral zone, certain losses occur. Some of the fine material is carried into deep water by currents and comes to rest in depths too great to be again transported back to the littoral zone by natural forces. No method is now available for estimating the quantity of material that is lost to offshore areas. Sand losses can occur into submarine canyons, but only the Partington Canyon appears to head close enough to shore for such losses to occur. No detailed study of this canyon head has been made; however, because of the relatively small drainage area upcoast from the canyon yead (Fig. 1), it appears that the upcoast supply of sand and therefore any loss into the Partington Canyon is probably relatively small. Losses by accretion at littoral barriers probably are negligible since

equilibrium conditions appear to be established at both natural headlands and man-made structures. Removal of sand for construction purposes also is a minor source of sand loss for the reach under consideration. The most important sand loss from the coast appears to be by wind action—particularly in the area between Pismo Beach and Point Sal (Fig. 1). No reliable methods are available for evaluating such losses, but the extent of dune formation in this region indicates that the loss by wind action is considerable.

ACKNOWLEDGMENTS

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Note.—Paper 2155 is part of the copyrighted Journal of the Waterways and Harbors Division, Proceedings of the American Society of Civil Engineers, Vol. 85, WW 3, September, 1959.

SELECTION OF DESIGN WAVE FOR OFFSHORE STRUCTURES[a]

Closure by Charles L. Bretschneider

CHARLES L. BRETSCHNEIDER,[1] M. ASCE.—The discussion by J. E. Chappelear was most appreciated, and his work contributes much to the design wave problem.

It is interesting to note the results presented in his discussion for the breaking index relations between H/L and d/L. His values are in very close agreement with those of the original paper for $d/L \geqq .09$, but for $d/L < .09$ the agreement is not too good. His values for all d/L-values are in very close agreement with the equation given by Miche (1944).*

$$H/L = 0.14 \tanh kd, \text{ where} \tag{1}$$

$$k = 2\pi/L$$

Eq. (1) tends to fail for $d/L < .09$ if the solitary wave theory is accepted as the upper limit for breaking waves in very shallow water. According to the solitary wave theory for very shallow water the breaking wave limit is given by

$$H/d = 0.78 \tag{2}$$

For very shallow water tanh kd tends to $kd = \dfrac{2\pi d}{L}$, whence from Eq. (1) one obtains

$$H/d = 0.14\ (2\pi) = 0.88 \tag{3}$$

which predicts the breaking wave height 13 per cent greater than that given by the solitary wave theory, Eq. (2). An attempt was made in the original paper to utilize Eq. (1) as a guide with particular emphasis on Bernoulli's equation and also wave data such that in water of infinite depth the breaking wave limit is $H/L = 1/7$ (instead of 0.14) and in very shallow water $H/d = 0.78$. In this respect Bernoulli's equation was utilized to obtain very nearly the other parameters η_b/H and L/L_A as functions of d/L.

A set of equations for the breaking wave limit which satisfies very nearly those results in the discussion (for $d/L \geqq .09$), and which satisfies very nearly the wave data of the original paper and also very nearly Bournoulli's equation for all d/L are as follows:

$$H/L = 0.124 \tanh kd \left[1 + 0.152 \tanh kd\right] \tag{4}$$

a. Proc. Paper 1568, March, 1958, by Charles L. Bretschneider.

1. Hydr. Engr. (Research), Beach Erosion Board, Corps of Engrs., U. S. Dept. of the Army, Wash., D. C.

* Miche, M. (1945): Movements ondulatoires de la mer en profondeur constante on decroissante. Ann. des Ponts et Chausees, tome 114, 1944.

$$\eta_b/H = 1 - 0.32 \tanh kd \tag{5}$$

and

$$L/L_A = 1.56 \frac{\tanh kd}{\tanh \frac{2\pi d}{L_A}} \left[1 - 0.177 \tanh kd - 0.059 (\tanh kd)^2\right] \tag{6}$$

where

$$L_A = \frac{gT^2}{2\pi} \tanh \frac{2\pi d}{L_A}, \text{ wave length by linear wave theory.}$$

THE SUEZ CANAL—ITS CHRONICLE AND BIBLIOGRAPHY[a]

Closure by Shu-T'ien Li

SHU-T'IEN LI,[1] F. ASCE.—To all who have contributed discussions, or communicated comments, the writer is deeply grateful.

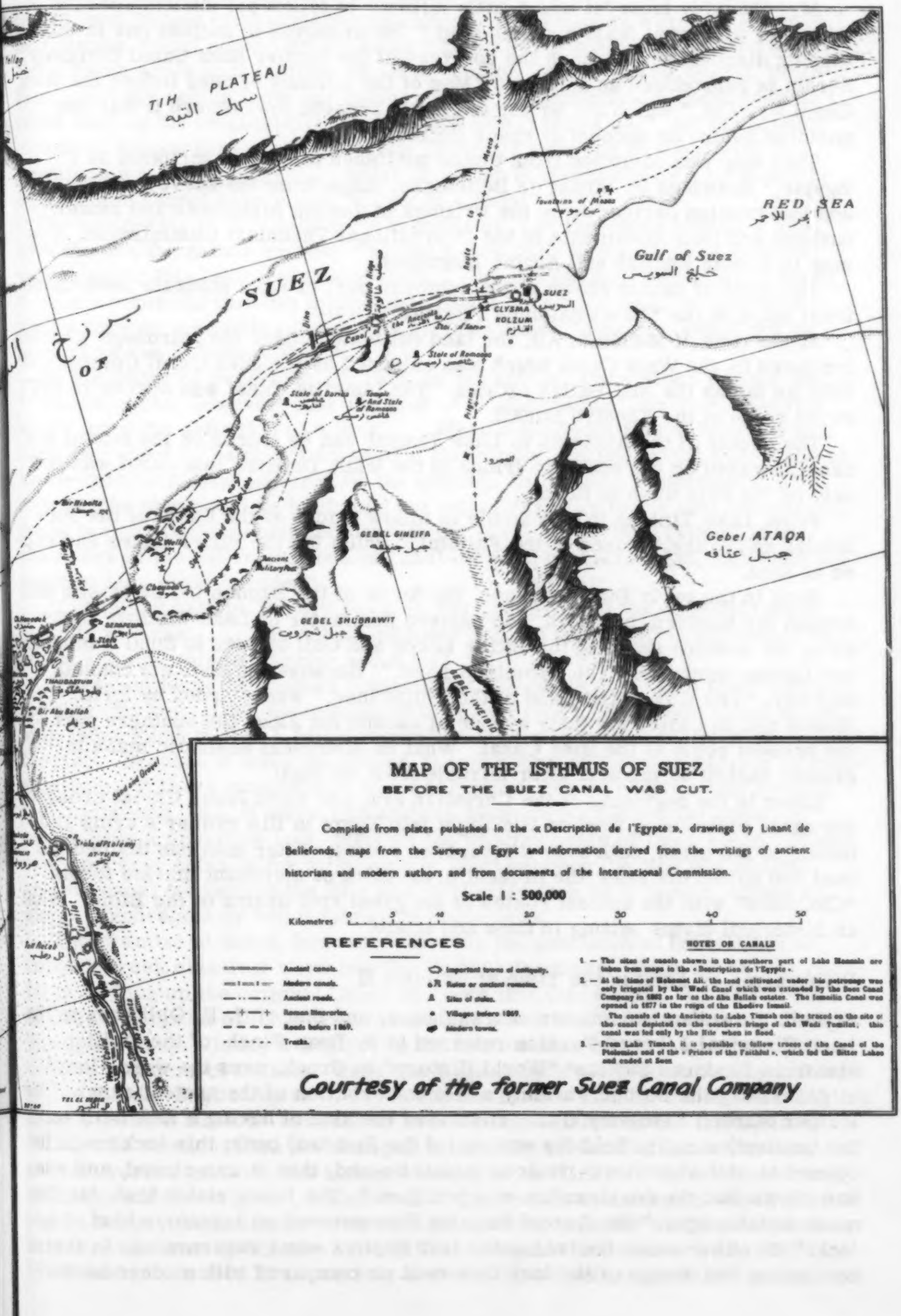
In addition to the discussions published, generous approval as well as encouraging comments were also widely expressed by many engineers, professors, and canal experts, from five continents. In alphabetical order, they are H. B. Blodgett, F. ASCE, Dean, College of Engineering, University of Nevada; C. Boillot, U. S. Representative, Compagnie Financiere de Suez, New York; Francois Charles-Roux, Honorary Chairman, Compagnie Financiere de Suez, Paris; Dr. N. A. Christensen, F. ASCE, Director, School of Civil Engineering, Cornell University; Dr. R. P. Davis, F. ASCE, Dean Emeritus, College of Engineering, West Virginia University; George Dunn, Retired Consulting Engineer, Dunedin, New Zealand; Sigurd Eliassen, M. ASCE, Consulting Engineer, Government Reconstruction Department, Oslo, Norway; W. A. Fairhurst, Consulting Engineer, F. A. MacDonald & Partners, Glasgow, Scotland; J. Georges-Picot, Chairman and Managing Director, Compagnie Financiere de Suez, Paris; Neal D. Howard, Executive Secretary, American Railway Engineering Association, Chicago, Ill.; Professor Shih-ta Hsu, F. ASCE, Chief Engineer, Shihmen Development Commission, Taiwan (Formosa), China; L. A. Loggins, F. ASCE, Chief Engineer, Southern Pacific Lines in Tex. and La.; F. M. Masters, F. ASCE, Consulting Engineer, Modjeski & Masters, Harrisburg, Pa.; S. S. Morris, F. ASCE, City Engineer, Capetown, Union of South Africa; H. R. Peterson, F. ASCE, Chief Engineer, Northern Pacific Railway; Colonel Stanley G. Reiff, Assistant Chief of Engineers for Civil Works, Corps of Engineers, U. S. Army; Dr. R. S. Rowe, M. ASCE, Chairman, Department of Civil Engineering, Duke University; Colonel H. C. Rowland, Jr., District Engineer, U. S. Army Engineer District, Wilmington, N. C.; A. L. Sams, M. ASCE, Principal Assistant Engineer, Illinois Central Railroad; F. R. Spafford, Assistant Chief Engineer, Boston & Maine Railroad; Professor H. S. Sung, Department of Civil Engineering, National Taiwan University, Taiwan (Formosa), China; Henry C. Tammen, F. ASCE, Retired Consulting Engineer, Howard, Needles, Tammen Bergendoff, Short Hills, N. J.

The large number of constructive comments, together with additional historical facts, geological explanations, technical views, and documentary informations, have greatly enhanced the subject matter. It is the writer's pleasure and privilege to attempt to present responses categorically in what follows.

a. Proc. Paper 1770, September, 1958, by Shu-T'ien Li.

1. Chf. Techn. Writer, Palmer-Baker, Inc., Mobile, Ala. Formerly Bd. Member, Grand Canal Impt. Planning Bd., Republic of China.


Gebel Moghara
جبل مغارة
Sand Dunes
CASIOS
El Gelis
Sand Dunes
QATIA
ISTHMUS
Cᵉ Chabrias GERRA
R. Mahamdia
Romani
MEDITERRANEAN SEA
Pelusiac Mouth
PELUSIUM
Tell Farama
THARU
PORT SAID
TELL TENNIS
Lake Menzala
DAPHNAE
HERACLEUS
El Gamil
Fort
Bahr


TIH PLATEAU
سهل التيه
RED SEA
بحر الاحمر
Gulf of Suez
خليج السويس
SUEZ
السويس
CLYSMA
KOLZUM
القلزم
ISTHMUS OF SUEZ
برزخ السويس
Fountains of Moses
Stele of Darius
Stele of Ramses
GEBEL GINEIFA
جبل جنيفة
Gebel ATAQA
جبل عتاقة
GEBEL SHUBRAWIT
جبل شبراويت
SERAPEUM
THAUBASTUM
Bir Abu Balloh
Stele of Ptolemy
Sand and Gravel
Cultivated land
TEL EL KEBIR
MAP OF THE ISTHMUS OF SUEZ
BEFORE THE SUEZ CANAL WAS CUT.
Compiled from plates published in the « Description de l'Egypte », drawings by Linant de Bellefonds, maps from the Survey of Egypt and information derived from the writings of ancient historians and modern authors and from documents of the International Commission.
Scale 1 : 500,000
Kilometres 0 5 10 20 30 40 50
REFERENCES
Ancient canals.
Modern canals.
Ancient roads.
Roman itineraries.
Roads before 1869.
Tracks.
Important towns of antiquity.
Ruins or ancient remains.
Sites of stelae.
Villages before 1869.
Modern towns.
NOTES ON CANALS
1. — The sites of canals shown in the southern part of Lake Menzala are taken from maps in the « Description de l'Egypte ».
2. — At the time of Mohamed Ali, the land cultivated under his patronage was only irrigated by the Wadi Canal which was extended by the Suez Canal Company in 1862 as far as the Abu Balloh estates. The Ismailia Canal was opened in 1877 in the reign of the Khedive Ismail.
The canals of the Ancients to Lake Timsah can be traced on the site of the canal depicted on the southern fringe of the Wadi Tumilat; this canal was fed only by the Nile when in flood.
3. — From Lake Timsah it is possible to follow traces of the Canal of the Ptolemies and of the « Prince of the Faithful », which fed the Bitter Lakes and ended at Suez.
Courtesy of the former Suez Canal Company

Map of the Isthmus of Suez Prior to the Present Canal

Mr. Shockley feels "it was a little difficult to follow the ancient courses of the canal without at least a sketch map." He proposes to include one in this closing discussion. Through the courtesy of the former Suez Canal Company, herein is reproduced an authentic "Map of the Isthmus of Suez before the Suez Canal was Cut," which the writer is glad to present now, though it was not possible before on account of space limitation.

This map was compiled from plates published in the "Description de l' Egypte," drawings by Linant de Bellefonde, maps from the survey of Egypt, and information derived from the writings of ancient historians and modern authors and from documents of the International Technical Commission. The map is in both English and Arabic languages.

The sites of canals shown in the southern part of Lake Manzala were taken from maps in the "Description de l'Egypte."

At the time of Mehemet Ali, the land cultivated under his patronage was only irrigated by the Wadi Canal which was extended by the Suez Canal Company in 1862 as far as the Abu Ballah estates. The Ismailia Canal was opened in 1877 in the reign of the Khedive Ismail.

The canals of the Ancients to Lake Timsah can be traced on the site of the canal depicted on the southern fringe of the Wadi Tumilat; this canal was fed only by the Nile when in flood.

From Lake Timsah it is possible to follow traces of the Canal of the Ptolemies and of the "Prince of the Faithful," which fed the Bitter Lakes and ended at Suez.

Back in the early Biblical times, the Route of the Exodus led by Moses was around the western, northern, and eastern perimeter of Lake Timsah, thence along the eastern shore of the Bitter Lakes and Gulf of Suez to Sinai Peninsula, and thence turning to "The Promised Land." Because the shortest coastal highway, "The Way of the Land of the Philistines," was guarded by forts, Moses had no choice for their avenue of escape but along the southern part of the present route of the Suez Canal. What an historical dramatic mass migration backed by miracle after miracle from on high!

Later in the beginning of the Christian era, our Lord Jesus Christ crossed the canal route twice first on His flight into Egypt in His mother's arms and thence to Nazareth, both over the caravan routes. After man has divided the land and united the seas, the present great ditch of merchant marine forms "CROSSES" with the ancient routes of the great epic drama of the Bible. What an historical divine setting in time and space!

Diaphragma or Lock in the Time of Ptolemy II

Both the author's quotation of translation, and that of C. H. Oldfather in the Loeb Classical Library Version referred to by Dean Finch, of the Sicilian historian Diodorus Siculus' "World History" in Greek, uses the word "lock" in describing the Second Ptolemy's last construction of the ancient canal. The former states: "Ptolemy II . . . conceived the idea of having a lock built in the (ancient) canal to hold the waters (of the Red Sea) back; this lock could be opened at will when it was desired to sail beyond, then it was closed, and use has shown that the construction was justified." The latter states that, "in the most suitable spot," the Second Ptolemy "constructed an ingenious kind of a lock." In either case, the translated text implies some vagueness as to the conception and design of the lock then built as compared with modern locks.

Dean Finch is to be credited for his extensive and exhaustive researches undertaken during the past decade into the ancient Babylonian, Egyptian, Greek, and Roman civil engineering culture. "Since locks were not invented," he reasons, "until some 1500 or more years after Ptolemy had passed on," Dean Finch wrote to the author on 19 November 1958: "I have waited, therefore, until my return from our country home in Connecticut to New York so I could look up the original Greek and check this translation." As his prudent classical scholarship put it, "Actually Diodorus referred to the device in question as a diaphragma, . . . which is properly translated as a partition or barrier and cannot have meant 'a lock in a canal'."

By comparing C. H. Oldfather's translation, "an ingenious kind of a lock," with the one the author quotes: "having a lock built in the canal to hold the waters back; this lock could be opened . . . to sail beyond, then it was closed," it seems plausible that two diaphragma might have been used, otherwise a single barrier, probably of stop logs as Dean Finch supposes, would not be termed as "an ingenious kind."

In his letter of December 29, 1958, to the author, Dean Finch further states: "I cannot agree, however, that any kind of a 'diaphragma', or single barrier, would meet the definition of a lock . . . When a single gate is opened the ship must be raised by pulling it up against the current caused by the difference in level—man or animal power does the raising, not water. Presumably, of course, the ship could be held until the entire reach up to the next barrier—if there is one—could be emptied down to the lower level, thus making lock chambers of the entire sections of canal between barriers. But, this does not seem to be a practical idea!

"I think the answer is very simple: the translator of the original Greek text of Diodorus . . ., was not an engineer and did not know the difference between a single gate or barrier and a lock."

Measured from the modern fundamental standards of a "ship lock" and the present-day prevailing nomenclature of "canal lock", the author could agree with Dean Finch in that a "diaphragma" should have been translated as a barrier. Because it could be opened, it was a kind of movable barrier or movable dam. If there existed only one diaphragma, the translation was clearly indiscriminate. But if there was another diaphragma not too far apart, it would form a primitive lock, meeting the requirements of Dean Finch. Moreover, because actually the difference in level between the Red Sea and the ancient canal connecting the Nile was negligible then as it is now, either a single diaphragma or two diaphragma could work.

The premise at issue, however, involves the precision of translation of ancient Greek classical work into the English language. Dean Finch was careful in looking up the original Greek to check this translation, but only from a modern standpoint of the English language, which has undergone many changes through history. Today's English is the continuation of the language of 5th-century invaders of Britain. This Old English up to C. 1050 A.D. was followed by the Middle English up to C. 1450, and the Middle then by the Modern English from a London dialect. In order to reach a fair verdict on the translation, taking into consideration the changes in meaning of the English vocabulary, archaic versus modern, the author has looked up "A New English Dictionary on Historical Principles", founded on the materials collected by The Philosophical Society, edited by Dr. James A. H. Murray with the assistance of many scholars and men of science. It has been compiled and edited from 1882 to 1927, and consists of 21 separately bound large volumes and 15,487 pages.

In this dictionary in its Vol. VI, Part 1, pp. 383-384, under "lock", one may find "II, A barrier, an enclosure," further under 7. "A barrier on a river, constructed so as to be opened or closed at pleasure"; and further under 9, "On a canal or river: A portion of the channel shut off above and below by folding gates (not a modern prevailing type) provided with sluices to let the water out or in, and thus raise or lower boats from one water level to another." The seventh meaning of the word "lock" in this dictionary exactly expresses the physical implications of a diaphragma that could be opened. If this meaning of the English word "lock" was prevalent in the time of the translator, Dean Finch's verdict that "the use of the word lock in this translation is evidently incorrect" would seem to be a modern view of a classical translation. Dean Finch's prudence in revealing the original Greek, and the writer's consultation of the monumental English dictionary on historical principles, complement to one thing that the original translation may there stand.

Dean Finch referred to the Grand Canal of China as 800 miles long, as having been begun as early as 600 B.C., and to the Chinese as never devised a canal lock. The writer feels privileged in having an opportunity to give authentic facts relevant to this comment, for he had been an organizing member of the Grand Canal Improvement Planning Board during the early 'thirties. The Grand Canal stretching from Peking to Hangchow has a length of 1,782 kilometers or 1,107 miles. Several segments are natural rivers; so is the section from Tientsin to Tungchow and thence by canal to the city wall of Peking, which is not situated on a river. Its initiation dates back to 600 B.C. just as Dean Finch stated. This was about the same time as Necho reconstructed and extended the ancient canal through Wadi Tumilat to beyond At-Tuku. Concerning the issue raised by Dean Finch that the Chinese never devised a canal lock, he is referred to the ten books in two volumes entitled "Hydraulic Problems of China," written by ten contemporary hydraulic experts of that country under the Chief editorship of the writer and published in February, 1937, by the Commercial Press, Ltd., Shanghai. Book VIII was devoted to "The Problem of Grand Canal Improvement," prepared by Mr. Hu-Chen Wang, formerly Chief Engineer of the Grand Canal Improvement Planning Board.

Hu-Chen Wang's exhaustive research shows that from 600 B.C. the dam type barrier had been used along the Grand Canal where the slope was too steep or where the discharge must be conserved; the diaphragm type barrier had been used since, where a movable type proved to be more advisable. Beginning from the Tang Dynasty early in the seventh century, locks of the sluice type were constructed at entrances of the Grand Canal where it crosses the Huai River and the Yangtze River. In the later part of the tenth century, early in the Sung Dynasty, locks of the double sluice type was introduced at the crossing of the Grand Canal with the Yangtze River. Its functional arrangement was in essence identical with modern navigation locks. Hu-Chen Wong gave the location of some 49 single- and double-sluice locks built from the beginning of the seventh century to the end of the seventeenth century (early in the Tsing Dynasty) along the Grand Canal and the Ling Ditch, the latter being the navigation connection between the tributaries of the Yangtze River and the Pearl River.

Philologically speaking, the Chinese nomenclature for "ship lock" uses the same characters for single-diaphragm, double-diaphragm, single-sluice, double-sluice, and modern types insofar as their functions are primarily for navigation. It so happens that they are in accord with the two definitions of

"lock" given in the New English Dictionary on Historical Principles. In 1950, when the writer was drafting the "Sino-English nomenclature of Hydraulic Engineering" for the National Institute of Compilation and Translation on his way to the United States on Board of M. S. Lightning, he coined the terminology according to historical usage. The nomenclature was later approved by a National Committee, proclaimed by the Ministry of Education as national standard terms, and published in 1952 by the Commercial Press, Ltd., on Formosa.

Dean Finch further traces the origin of lock and its forerunner, the sluice gate used in irrigation, from Italian records, showing that hand-operated sluice gate dates back not earlier than the twelfth century, nor boat locks earlier than the fourteenth century—the early days of Renaissance. If he had chosen the ancient usage of the word "lock" as the seventh meaning defined in the New English Dictionary on Historical Principles, he could have placed the origin of boat lock at the time of the Second Ptolemy in the first half of the third century B.C.

Hans Straub in his "Die Geschichte der Bauingenieurkunst" (Verlag Birkhäuser, Basel, 1949; English Translation by E. Rockwell, "A History of Civil Engineering," an outline from ancient to modern times, Leonard Hill Limited, London, 1952, pp. 130-131), appears to confirm Dean Finch's findings. To quote Straub:

> "With the construction of the Atlantic and Channel Ports which are particularly exposed to the tides, the most important engineering works were the locks. To overcome the level differences of inland waterways, such installations were already in use since the Renaissance. Leonardo da Vinci had not, in fact, invented them as is sometimes alleged, but he may have perfected them through the development of an improved gate design. Applied to the tidal ports, the locks made it possible to retain, also during the low tide, the depth of water required for the sea-going vessels anchored in the inner basin . . . ."

In the Far East in China, hand-operated sluice gate had been introduced in irrigation in the fourth century B.C. since the diversion of the Chang River to irrigate the district of Yeh during the period of "Warring States" (403 to 221 B.C.; near the end of the Chou Dynasty, 1122 to 249 B.C.). This was the advent of the sluice-type of boat lock constructed later in the seventh century during the early Tang Dynasty.

Dean Finch concludes his discussion by turning back to Diodorus. His pondering on "lock" indicates prudent classical scholarship—a rare scientific attitude among engineers.

Natural Selection of Route and the Human Efforts

Dr. Chou begins his discussion with this topic. Its raison detre underlies practically every open canal, lock canal, even lateral canal, and nonetheless in canalized rivers and improved waterways. In between "natural selections of route" and "the human efforts," there seems always a third factor dictating this kind of project and that is "transportation demand." Nature provides the physical factor; transportation demand justifies the economic. Man was ordained to "have domain over all the earth" and "subdue it," and hence he digs the canal or improves a natural waterway when it becomes necessary.

Dr. Chou attributes the realization of the Canal to Napoleon and de Lesseps. Suffice it be so apparently, the inspiration to construct the modern Canal was originated from Count de Saint Simon who thought of canals at Suez and Panama as primary factors in the "regeneration of the world." De Lesseps seized the inspiration, had persistent faith in it, and hence there resulted this waterway for merchant marine of ocean commerce of all flags.

Viscount Ferdinand Marie de Lesseps

Mr. Millecam appropriately calls attention to the life of Viscount Ferdinand Marie de Lesseps; he regards him as a diplomat rather than an engineer. In the course of preparing the paper, it was fully realized that the chronicle of the Canal would not be complete without a narrative of de Lesseps, two-thirds of whose life had been interwoven with the history of the present Canal. However, on account of space limitation, the author had to adhere only to the main chronicle of the Canal itself and withhold the section narrating the life of de Lesseps from 1805 to 1894.

Much of de Lesseps' earlier life has been accounted for in connection with Saint Simon's inspiration, his promotion of the Suez Canal project starting at an age of 27 in 1832, his organization of the Compagnie Universelle du Canal Maritime de Suez in 1858, his commencing in the excavation of the Canal in 1859, his resorting to machinery during the construction period, his persistence in raising the necessary funds to meet the mounting demand, and his completion of the Canal in 1869 at the age of 64. It was reserved to a later paper dealing mainly with social, economic, and international aspects, that his tenacious strife to overcome the opposition of Lord Henry John Temple Palmerston, Prime Minister of the United Kingdom during 1855-1858, and of the "Sublime Porte," sultan of Turkey at Constantinople, the suzerain to the Viceroy of Egypt, would be treated.

Mr. Millecam calls attention to the remarkable book entitled: "Ferdinand de Lesseps—Le Diplomate—le createur de Suez," by George Edgar-Bonnet. But the writer could not agree with attributing de Lesseps as "le createur de Suez" for Christian reasons, because no one except God could be regarded as the creator of Suez.

Nor could the writer agree with Mr. Millecam in limiting de Lesseps as a diplomat but not an engineer. Apparently Mr. Millecam is qualifying him by his early calling not by his life-long achievement. This point may be elucidated by citing a notable example in the United States. The writer refers to the late George S. Morrison, Past President of ASCE. His career was undoubtedly one of the most unusual in the annals of engineering just as de Lesseps'. Morrison was graduated from law school in 1863, practiced law with a famous firm in New York, disliked thoroughly the equivocal requirements of legal practice, turned to bridge work in 1887, and built sixteen remarkable bridges over the greatest river system of the United States, ten over the Missouri River, one over the Ohio River, and five over the Mississippi River. He was a figure from law practice to a top-ranking bridge expert without any formal engineering education. If he had been regarded as a lawyer, not an engineer, he would not have been elected during the past to the President of ASCE, one of the most honorable offices of the American engineering profession. Similarly, unless de Lesseps had been regarded as an engineer of exceptional technico-scientific attainment, the Frenchmen would not have elected him to member of the French Academy and the Academy of Sciences, the highest honors a French man of science may receive.

In the strict sense, nor was de Lesseps a diplomat in spite of the fact that he served in the French consular service and first went to Egypt in 1832 as vice consul at Alexandria. Consular service, as is inherent in its function, constitutes an organized body of agents maintained by a government at foreign ports and trade centers to protect its nationals and their interests, especially in commercial affairs. For administrative purposes, many countries unite consular and diplomatic services in the Department of Foreign Affairs (Foreign Office, or Ministry of Foreign Affairs, or Department of State), as the United States did by virtue of the Rogers Act of 1924.

The writer, however, can agree, in a broad sense, to call de Lesseps a diplomat, if at the same time he is respected as an engineer, a promoter, an organizer, and also an author. He had contributed much to the engineering literature especially on his master project, the Suez Canal. Hereunder is a list of nine volumes of his published works on the Suez Canal as far as the writer is able to ascertain:

1. "Percement de l'isthme de Suez;" 1885, 280 pp.; H. Plon, Paris; First of a series of six volumes of documents.
2. "New Facts and Figures Relative to the Isthmus of Suez Canal;" 1856, 223 pp.; E. Wilson, London.
3. "Inquiry into the Opinions of the Commercial Classes of Great Britain on the Suez Ship Canal;" 1857, 147 pp.; J. Weale, London.
4. "Lettres, Journal et documents pour servir a l'histoire du Canal de Suez;" 1875-81, in 5 volumes; Didier et Cie, Paris.
5. "The History of the Suez Canal;" 1876, 89 pp.; W. Blackwood and Sons, Edinburgh and London.

Having firm faith in Saint Simon's inspiration about canals at Suez and Panama as primary factors in the "regeneration of the world," de Lesseps turned to begin his push of the Panama Canal in 1874, the second profitable year of the Suez Canal, when he was 69 years old. His fortune was assured, his name a popular household word throughout France, his fame worldwide. The French became interested in the isthmus canal during 1874-76, and were seized with ambition to follow the Suez Canal with one at Panama. In 1879, the "Compagnie Universelle du Canal Interoceanique" was organized to dig the ditch with Ferdinand de Lesseps elected as its President. De Lesseps insisted that this one should be like the Suez ditch, at sea level, without locks. In 1883, large scale work began on the excavation of an estimated 157 million cubic yards of material. But the tropical climate, with its torrents of rainfall, miasmata and deadly fevers, led by yellow fever, failed him. He gave up in 1888; the company bankrupt, his fame sullied with recriminations, his name anathema, his fortune gone irrecoverably, and his age 83. The French "Affair" ended 1889, due mainly to disease among the workers. Later a payment of 40 million dollars was made by the United States to the French for their interest; the Panama Canal became the property of the United States.

If de Lesseps had had the service of Dr. William C. Gorgas of the U. S. Army Medical Corps who stamped out "yellow jack" on the Isthmus and made construction possible he would have achieved success at Panama too. His failure at Panama cannot be attributed to his sea-level plan. This is clear by referring to the Society's Transactions, Vol. 114, 1949, Paper No. 2378, "Panama Canal—The Sea-Level Project, a Symposium," pp. 607-906.

After he gave up the Panama Canal Project at an advanced age of 83, he had the consolation that his son Charles took on himself, as much as he could,

the opprobrium of his failure, and that his family shielded him from the violence that assailed his reputation and attacked his honor, though such never happened in England. Almost in another world, therefore, esteeming his achievements as perhaps those of another man—another Saint Simon protégé—he haunted his garden and paced out his remaining years. He died in 1894 at the age of 89. More than halfway at least, on the ocean routes of modern international commerce, he had advanced the "regeneration of the world."

France as a nation was just in having conferred upon de Lesseps the honor that was eminently due him. The French Academy elected him a member, as did the Academy of Sciences—the highest honors the French can give a man of science. He was also decorated with the Grand Cross of the Legion of Honor.

Despite Palmerston's opposition to his Suez Canal Project, he became a British hero and remained so even after his failure at Panama. He was made a "Citizen of London" and decorated with the Star of India. He received the freedom of the City of London—the old City, under the Lord Mayor, with its guild-halls and proud traditions and formal charter. Not only England never besmirched his genius with his failure at Panama, but also the Britannica remarks: "His great gifts, coupled with his supreme unselfishness and social charm made him everywhere respected." History pays him the same respect.

De Lesseps' genius, tolerant nature, fairness toward men, unselfishness, and persistence in advancing the "regeneration of the world," should command eternal respect among mankind! His statue at the head of the Canal at Port Said recalls him worthily.

Initial Size of Canal Facilities

Dr. Chou, in discussing evolutionary growth, brings to a climax by stating: "Structures of international standing should be designed with generosity rather than with mediocrity, unless there is sound basis to be otherwise."

The 1856 plan of Negrelli de Moldelbe, which became essentially the plan of the International Commission, had a canal section of 8 m (26.25 ft) in depth, 44 m (144.36 ft) in bed breadth, and 80 m (262.47 ft) in water surface width. It was generously ample for the then potential traffic. However, after the first issue of shares in 1858, the Supreme Works Council decided in August, 1859, to retain the depth, but to reduce the surface width and the theoretical bottom width. Even so, the original cost estimate of 200 million gold francs was exceeded by 220 million francs by 1869 when the initial depth of only 5.20 m (17 ft) to 5.50 m (18 ft) was excavated for opening to traffic. The initial project had not been completed until 1875. It was obviously difficulty of financing that had to overshadow any recognized sound basis. De Lesseps faced the situation with individual audacity, but with little help from the French government. Had his plan been more generous, the probability for realization at that time would have been remote.

De Lesseps functioned international responsibilities with a French joint-stock company. It was not an international project financed and backed up by a family of nations. On the contrary, the British empire and the Ottoman empire exercised opposition and suppression which will be revealed in the later paper.

However, de Lesseps' fruit has been dynamically growing all the time with ocean commerce. In contrast with the phenomenal growth of the Suez Canal are such generously built monuments of the ancient kings and emperors, out

of state treasury and by national labor, as the Tower of Babel in the City of Babylon, the Greatest Pyramid of Khufu (or Cheops) at Gizeh (or Giza) on the Nile opposite Cairo in Egypt, and the Great Wall of China. They have remained static despite their generous proportions.

De Lesseps built a revenue-producing, economic facility which has grown by regenerative financing. This turnpike of the seas will continue its pace of growth in the future even at a faster rate as revealed by the Report of Ebasco Services, Incorporated, of New York.

Geological and Soils Aspects

Perhaps in no other civil engineering works are the maintenance and operation problems so much interwoven with geological and soils aspects as a sea-level earth canal for ocean traffic. These aspects were necessarily touched upon only in a sketchy way in a paper of engineering chronicle. Thanks are due to two eminent engineering geologists, Professor Morris and Professor Cleaves, for their illuminating discussions.

In addition to confirming the very brief geological account of the author, Professor Morris summarized the geological periods (systems of rock), and epochs (series of rock), of sedimentary rocks in the southern segment of the canal zone. His geologic sketch helps to clarify questions concerning the geology of the Canal route. He refers to the quarried marine limestone for the "Pyramids of Gizeh" and the natural hill of "Sphynx," both near by Gizeh (or Giza) on the Nile opposite Cairo, Egypt. The greatest pyramid is called the Pyramid of Khufu or Cheops, who was king of ancient Egypt, flourished 2900 (?) B.C., founded the IVth dynasty, and built this mammoth at Gizeh. Sphynx is more orthodoxly spelled as Sphinx. Egyptian sphinxes are recumbent figures, usually with men's heads. Most famous of all is the Great Sphinx of Gizeh, a colossal figure carved from natural rock, guardian of the Nile Valley. The marine limestone of the Eocene epoch, with which the Pyramids of Gizeh were built, is of the Tertiary period of the Cenozoic era.

Using the geologic time scale most generally accepted in the United States, with duration in years based on radioactive disintegration, Professor Morris' summary consists of two eras (1) the Cenozoic (to represent the modern life), and (2) the Mesozoic (the mediaeval life). The Cenozoic era consists of (a) the Quaternary period from 25,000 yr to 2 million yr, and (b) the Tertiary period of about 68 million yr, which embrace the Pliocene, Miocene, and Eocene epochs mentioned by Professor Morris. The Mesozoic era has as its latest the Cretaceous period, also mentioned by Professor Morris, which has been estimated at about 180 million yr.

While Professor Morris' discussion is chiefly focused on stratigraphy, historical and spatial, Professor Cleaves has devoted his to the engineering significance of geological and soils aspects, and to construction materials. Though his querries are not entirely covered by, but status quo answers could be found in, the paper of Max Bahon presented at the Seventeenth International Navigation Congress held in Lisbon in 1949 and that of M. Leroy at the Eighteenth in Rome in 1953, both being "about the particularly arduous technical problem the (Suez Canal) Company had to deal with in order to protect the Canal banks from erosion caused by the passage of large vessels," as Mr. Millecam put it. Abundant light may also be found from laboratory tests as reported in ASCE Proceedings Separate No. 613 (February 1955) by Paul A. Blanquet, and in a detailed report in Annales des Ponts et Chanssees

(November 1957-February 1958) by Suquet, Barbier, and Gamot. ". . . these laboratory studies have made it possible," according to Mr. Blanquet, "to establish as correctly as possible the regulation transit speeds so as to avoid deterioration of the banks and facings" and "to select the most appropriate phototypes of cross-sections suitable to the various sectors."

Professor Cleaves touched the question of quarry limestone. Good-quality material of its kind is not entirely lacking in that broad region, but unfortunately the distance from some of the works to the quarries from which suitable stone for the bedding could be obtained makes it prohibitive in cost. As far back as the construction of the jetties at Port Said before the Canal was opened to traffic in 1869, the use of stone from the quarries at Mex, near Alexandria, proved to be so. It was then substituted by artificial blocks made of lime mortar, using the sand at Port Said as the basic material. These lime-mortar blocks, lasting for more than 30 years under the exposure of sea waves and salt water until the beginning of the Twentieth Century, had not shown a bad performance record for a non-hydraulic binding agent. Since the beginning of this century good-quality stone has been taken from the Ataqa quarries on the western shore of the Suez Roads.

The use of lime mortar for making artificial blocks shows another evidence that de Lesseps was confronting constantly with the difficulty of raising the necessary money to meet the mounting demand of the construction cost. Realizing that the first Portland cement factory on the European Continent was established in France at Boulogne as early as 1840, and that the construction of the initial Canal had been from 1859 to 1869, de Lesseps would have used Portland cement if he had not been handicapped by financial difficulty.

The various types of revetments used in the past, namely: quarry stone, steel sheet piling, concrete blocks, reinforced asphalt carpets, etc., should have provided by now sufficient service records to choose the best type or types of revetment for different sectors based on the specific merits of each type to suit local conditions.

Despite that the Canal, except in the Lakes, is lined on either side by revetments, "slip-out" slides occur, though generally confined to the mid-segment where the highest ridge at El Gisr is some 52 to 56 ft above sea level. From El Gisr northward, the shallow depressions are even 3 ft or so below sea level; and from there southward the Canal traverses through Lake Timsah, Greater Bitter Lake, and Little Bitter Lake.

As slides have appeared to be not uncommon, a modern geological and soils engineering investigation as outlined by Professor Cleaves should prove to be conducive to effecting genuine maintenance savings along the Canal. He has also pointed out techniques to be followed in several categories of studies.

Judging from the necessity of constructing 75 miles of bank facings as already planned for the Ninth Program of Works, as revealed by Mr. Blanquet, the magnitude and importance of the investigation would justify a team of world-wide experts with one or two in each of the fields representing geology, geophysics, petrography, clay-technology, soil mechanics, engineering materials, and canal and harbor engineering. Even if it be the case that a great amount of work along this line has been done, a fresh review coupled with further investigations from a new approach, by new experts may shed new light through reappraisal and thereby produce better results.

Mr. Bostian's discussion also dwells for a while on soil mechanics and its problems associated with the Canal. In essence, he and Professor Cleaves complement each other.

Soil Conservation Program

With the unique background of having had hydraulic engineering research and practice in Continental China, on British isles, in Free China on Formosa, on Ceylong, and in the Continental United States, Dr. Chou attributes the chief trouble in heavy maintenance to sandy and unstable earth in the desert region, subjected to disturbance of viscous fluids; and to proximity to the muddy Nile and the Sahara Desert bringing in material from a great distance. He suggests that any scheme to bring down the recurring perennial expense requires the extension of the Canal jurisprudence beyond its present periphery both physically and categorically. Although terrestrial physical control as extensive as implied by his suggestion can not yet be realized, the Nationalization has at least but one merit in enabling it possible to launch effective conservation program in adjoining zones and contributary districts to meliorate the natural conditions and to minimize the threat of silting and deteriorating.

Dr. Chou then brings out the measures of soil conservation. The U. S. Department of Agriculture has long been pioneering in soil conservation. During the writer's term as Deputy President of the Yellow River Commission from 1943 to 1947, a program to conserve soil over the entire watershed of the Yellow River and its tributaries was under execution. There are diverse ways to achieve the objective under different local topographic, agricultural and forestation conditions. Though space does not permit to ponder further, soil and/or water conservancy can not be separated from any waterway maintenance if the maximum coordinated benefits are to be derived.

Irrigation and land reclamation have been carried out to some extent on the west side through the territory served by the sweet-water canal. Dr. Chou in this regard is concerned with that the agriculture of this swampy and sandy area can hardly compete with the alluvial Nile. Without a sizable agriculture population inhabiting in the Isthmus, there is no fundamental necessity nor supporting agrarian labor for any irrigation or land reclamation scheme. The cheapest way is to determine the most adaptable grass, and spread its seeds after rain by aeroplane.

Dr. Chou considers the silt from the very turbid water of the Nile effluence as induced by the gentle movement into the Red Sea. But the paper, on the contrary, states a weak current up to 1.9 ft per sec is toward the Mediterranean. According to de Thierry, the variation of the cross-section and silting due to this current is negligible.

Dr. Chou and Mr. Bostian both refer to the silting problem; the former proposes settling bay or pool with some coagulants as a possible solution. Economy does not seem to justify the use of coagulation methods. It appears rather more economical to have control works to allow high silt-bearing waters of the effluents of the Nile to flow into swampy low lands to settle by the gravity of silt as velocity vanishes. This method converts swampy waste into fertile fields with minimum labor and cost. When the writer was Chairman of the Technical Committee of the Hai Ho (Tientsin Seaway) Improvement Commission from 1929 to 1933, a scheme of control works was executed to divert the heavy-silt-bearing waters of the Yung Ting Ho, an influent of the Seaway, to an adjacent low-lying swampy expanse and to bring back the clear water to the Seaway for deepening the channel by scouring action especially during low water seasons.

The Suez Canal zone situation is different. Its conservation problems have to be solved by a distinctive scheme utilizing the available physical phenomena

to gradually convert the desert and swampy waste into a productive green country.

Canal Cross-Section and Vessel Characteristics

Mr. Blanquet brings out his first major problem bearing on the relationship between the cross-section of a canal and parameters of the vessels using it—such as draft, width, and speed. To these the writer would like to add such other parameters as length, displacement, configuration, and total immersed longitudinal surface of the vessels; surface slope, and better still hydraulic or energy gradient, of canal water; physical constants of this water such as unit weight (w), density (ρ), coefficient of viscosity (μ), and kinematic viscosity (ν); and last but not the least, the composite dimensionless parameters of Froude Number (N_f), and Reynolds Number (N_r). Besides these for the vessel and the water, those pertaining to the canal cross-section would have to include ratio of breadth of waterway to beam length of vessel, ratio of depth of waterway to draft of vessel, ratio of waterway cross-section to vessel cross-section, and the geometric form of the waterway cross-section. And more over, in no small measure are the physical phenomena of the material in suspension and density currents of the canal water.

The fine experimental investigations conducted by Mr. Blanquet in the Bassin des Carenes du Ministere de la Marine in Paris and in the Neyrpic Hydraulic Laboratories in Grenoble, France, deserve most careful perusal by any student of waterway engineering. Besides his paper on "Scale Model of the Suez Canal" published in the Society's Proceedings of February, 1955, as Paper No. 613, Messrs. Suquet, Barbier, and Gamot's detailed report on these important experiments as published in the November 1957 to February 1958 issue of the Annales des Ponts et Chaussees constitutes a valuable contribution to the hydromechanics of waterways. Mr. Blanquet has set forth the principal conclusions and their applications to establishing the regulation transit speeds, and in selecting the most appropriate prototypes of cross-sections suitable to the various sectors.

During the early 'thirties, the writer was entrusted in the capacity of Vice-President of the Board of the First National River Hydraulic Laboratory of China, in charge of establishment, development, and research. That laboratory, partly indoor and partly outdoor, was the largest of its kind in the Far East at that time.

In ship-canal-model experimentation, the water suffers inappreciable compression during the flow. Hence, the elasticity forces may be neglected. Except in the case of extremely small models, surface tension may also be disregarded. Thus, the forces left to be considered are pressure (F_p), inertia (F_i), viscosity (F_v), and gravity (F_g). By deleting the hydrostatic pressure force, and denoting "prototype" and "model" respectively with subscripts "p" and "m", the dimensionless parameters of N_f and N_r may be written into the following equalities:

$$\left(\frac{F_i}{F_g}\right)_p = (N_f)_p = \left(\frac{V^2}{\ell g}\right)_p = \left(\frac{V^2}{\ell g}\right)_m = (N_f)_m = \left(\frac{F_i}{F_g}\right)_m$$

$$\left(\frac{F_i}{F_v}\right)_p = (N_r)_p = \left(\frac{V\ell\rho}{\mu}\right)_p = \left(\frac{V\ell\rho}{\mu}\right)_m = (N_r)_m = \left(\frac{F_i}{F_v}\right)_m$$

The solution of these two simultaneous equations yields

$$\frac{l_p}{l_m} = \left(\frac{\nu_p}{\nu_m}\right)^{\frac{2}{3}}$$

which shows that the selection of the model scale should be accompanied by a corresponding kinematic viscosity of the fluid for the experimental channel. Either an appropriate economical fluid must be found for the supply, or instead, if water is used, the model scale must be unity, which would imply full-scale model. This peculiar a priori fact has entailed an almost insurmountable obstacle to ideal dynamical similarity for ship-canal models. It becomes imperative to approach the experimentation with an expedient compromise by resorting to the same Froude Numbers in prototype and model, and by adjusting the observed results by auxiliary experimental data dependent upon the Reynolds Number. Since the hydraulic conditions resulting from inertia-viscosity forces are small as compared with those resulting from inertia-gravity forces, the compromised approach is admissible.

While in ship-canal-model experiments, both ship and canal characteristics interact onto each other, the waterway parameters are the independent in towing basins, but those of the ship become the independent in the case of determining (1) the best regulation transit speeds to avoid deterioration of the banks and revetments, and (2) the most appropriate prototypes of cross-sections.

As the results of Mr. Blanquet's laboratory studies agree with those of direct observation programs and with measurements taken in the Canal itself, these studies should prove to be very valuable, too, in proving correlated criteria for design purposes, though they might have been conducted for the desiderata of maintenance and improvement.

The writer deems that Mr. Blanquet's experimental studies should have answered Dr. Chou's calling attention to scale model tests, except for the latter's proposed desilting facility. The same studies also lead to appropriate answers to Mr. Bostian's discussion of the Canal cross-section.

Wave Action in Relation to Canal Banks

Mr. Bostian dwells at length in the subject of waves and their disturbing action in relation to canal banks. Here lies a realm embracing scientific interest as well as engineering significance that may be approached either from a mathematical gymnasium or with an experimental technique. Lest this closure exceeds allowable limit of space, the writer shall not embark a down-to-the-earth discussion which would not bring fruit unless it is given an exhaustive treatment. But, instead, by way of response, the writer wishes to refer Mr. Bostian to a series of valuable research reports along this line made available by the Beach Erosion Board, Office of the Chief of Engineers, Corps of Engineers, Department of the Army. These reports are issued as Technical Memoranda which have reached No. 111 up to May, 1959. Their investigations were made on shores, coasts, beaches, and waterfront structures, but are applicable to sea-level canals. Much valuable inference could also be obtained by scanning through Paper No. 2378 entitled: "Panama Canal—The Sea-Level Project, A Symposium," ASCE Transactions, Vol. 114, 1949, pp. 607-906.

Traffic, Past and Future

Mr. Jos. Millecam observes that to the author's statement: "Thus it (the Canal) constitutes the life line of Western European maritime countries, particularly Britain and France," should be added "'the Netherlands,' which country," he states, "kept up an intense maritime traffic with their East India colonies (at present Indonesia)."

An examination of the traffic records of the Suez Canal does support Mr. Millecam's comment. In the period from 1879 to 1890, the proportion of total net tonnage through the Canal according to flag was indeed in the order of British, French, Dutch, etc. Though from 1891 to 1910 Dutch was in the fourth place, she regained the third place for most of the time from 1911 to 1920. She then advanced to the second place from 1921 until 1929, but dropped again to the third and fourth places in equal number of years from 1930 to 1947. Thereafter until 1951, she fell to the sixth place. In 1955, she apparently sunk to the seventh place with British, Norwegian, Liberian, French, Italian, Panamanian ahead of her. But in reality she is still in the fifth place, because ships with Liberian and Panamanian flags belong to merchant marine interests of many different countries. They simply register their companies in Liberia and Panama for incorporation and taxation reasons.

British merchantile marine has always held the first place as a Canal user from 1869 up to the present. During the period between 1870 and 1880, there was 76.1 per cent of the transiting vessels which flew the British flag. It reached 79.5 per cent in 1918; but "the carrier of the oceans" fell to 28.3 per cent in 1955, indicating the shrinkage of the British empire.

It is interesting to note the change in hands of the second largest user of the Canal according to flag. From 1870 to 1890, it was French. Between 1891 and 1914, German came forward, reflecting its gain in ocean commerce up to World War I. In 1915, 1916, and 1917, Dutch, French, and Italian respectively, held the second place. Having no suffer during World War I, Japanese advanced to the second place from 1918 to 1920. Thereafter through 1929, Dutch flag was in the second place. Between 1930 and 1934, the revival of Germany came forward again. From 1935 to 1940, Italian advanced to the second place, but the ships included Mussolini's war vessels during his aggression against Ethiopia. In the earlier years of World War II from 1941 to 1942, Greek had the second place, while during the later part of World War II and early post-war years from 1943 to 1949, American ships held the second place. As a consequence of its participation in tanker traffic, the Norwegian flag has taken the second place since 1950.

Very much enhancing to the discussions is the second problem Mr. Blanquet brings out which "relates to the most recent forecasts of traffic-evolution made by the Concessionary Company and to the conclusions they inspired." This especially complements the author's paper in which the studies made by the Ebasco Services, Incorporated, of New York was only sketchily introduced.

The Ebasco Report has based its study on six optimistic assumptions for the world's economy. Among them, it was assumed that "the policies of the under-developed nations would be influenced by a desire to develop their resources and their industry for the benefit of their domestic economy." This is only a moderate view from the New York skyscrapers. Those familiar with oriental ambitions would not be surprised to note Dr. Chou's comment: ". . . It may be not far off to when another source of tremendous potential for the traffic through the Canal will come, when the growing industrialization of the

Southeastern Asian and Far Eastern countries increases the volume of exchange from Europe." He added: "this is another challenge to the Canal."

Ocean commerce is so interwoven with international situations that whether the Ebasco assumptions and their conclusions are too modest or otherwise will be borne out by 1972 up to which year the forecast was made.

The former Suez Canal Company is to be complimented for its projected Ninth Program of Works whose plans were completed in the Eve of Nationalization. It is hoped that such works would be executed, with no less scale and no longer lag, by the present Suez Canal Authority.

Canal to Grow with Tankers

Mr. Leslie challenges the author to express his views on the economic aspects of the Canal and, in particular, on whether the Canal can keep pace with the rapid increase in size of modern tankers, or whether that commerce will go outside, and the Canal lose the great bulk of its present commerce.

While the economic aspects of the Suez Canal, past and future, will be treated among other things in a later paper, the governing criteria of the economics of any waterway in any country must be approached from a coordinated transportation system of an entire nation if she is more or less isolated, or from an internationally coordinated transportation system of a community of nations if their economy is interdependent. By "coordinated transportation system" in a nation or in a community of nations is meant an efficiently planned overall transportation system including inland waterways, coastal and ocean shipping lines, railroads, highways, and air lines.

Just how well the coordination of an overall transportation system could be effected under the stimulus of free competition within a system of free enterprises, it deserves the best attention of statesmen, lawmakers, engineers, and economists. In the ultimate analysis of any transportation project, waterway or otherwise, long-range, true economy can only be assured if it is justified from a coordinated transportation point of view, rather than from a particular point of view.

Returning to the question of tankers transiting the Suez Canal, it is obvious that their only competitor is the "big-inch" pipe line. Whether the bulk fluid commodity of crude oil or petroleum products flows by gravity or by pumping pressure, the entire transmission-transport process from source to destination requires trans-shipment handling facilities, pumping equipment, and storage installations, besides the pipe line.

Although, after World War II, the 30- and 31-inch Trans-Arabian pipelines have been completed to carry oil from the wells in Saudi Arabia 1200 miles to Sidon on the East Mediterranean in Lebanon, where it is stored for trans-shipment by tanker to Western European countries; the pipe lines, however, delivered only a total of 117,988,503 barrels in 1955 which is about one-third of the total production of crude oil from the fabulous Ghawar field in that year. During the Middle East Emergency, by pumping lighter crude oil, placing temporary additional pumps on the line, and raising the pumping pressure, Trans-Arabian Pipe Line Company expanded their throughput by some 25,000 barrels daily in the first half of 1957. If the increased throughput were maintained year round, those pipe lines would be able to deliver about 127,000,000 barrels. Comparing this figure with the over 600,000,000 barrels of crude oil and products transported by those Jersey (Standard Oil Company of New Jersey, a holding company) companies alone having shipping operations during

the single year of 1956, the capacity of trans-Arabian pipelines is only about one-fifth of the shipping capacity of the tanker fleet of Jersey.

While the existing trans-Arabian pipe-line capacity is far from adequate to supply Western European countries, the tense situation of Middle East does not encourage the international oil companies to further invest in the provision of additional transmission capacity, unless sabotage threat is permanently removed from the horizon.

Marine transportation is flexible in route and versatile in use. Besides the large fleet of self-owned tankers, the petroleum industry has used many special-service vessels, numerous coastal craft, and, in addition, chartered a large number of tankers from outside sources.

The undesirability of being dependent upon charters from outside interests in time of emergency has led to the industry's policy of assuring a substantial coverage of necessary marine transportation through owned tonnage. This policy is reflected in an accelerated program of tanker building. In 1957 alone, the industry had on order, or under construction, nearly one hundred ocean tankers, of which, owing to the trend toward larger, more economical vessels, about a score of them are of the 46,000-dead-weight-ton class, some even over 100,000 tons.

Thus, the tanker traffic will continue to increase in the future. As to the forecasts of traffic that would transit through the Suez Canal, the writer wishes to call Mr. Leslie's attention to the second part of Mr. Blanquet's discussion based on the Ebasco Report.

Measured by the 1955 net earning realized by the former Suez Canal Company, the Canal revenue could bring to Egypt through the management of the Suez Canal Authority a net revenue not less than 6 to 7 per cent of the total treasury revenue of Egypt. Unless the Canal can keep pace with the rapid increase in size of modern tankers, the purpose of Nationalization would be defeated economically. The writer believes Egypt will struggle with the increase in size and number of tanker fleet.

Comparison with the Panama and Other Canals

Mr. Leslie notices major differences and common problems between the Suez Canal and the sea-level Cape Cod Canal in New England.

Governor Potter has contributed an invaluable and most interesting discussion by setting forth the parallel engineering purposes and common heritage of the Suez and Panama Canals, the success of de Lesseps at Suez and his defeat at Panama mainly due to disease, the international treaties and Congressional Acts relative to the Panama Canal, and the physical differences between the two canals.

Besides those pointed out by Governor Potter, it is of interest to note the additional features that are in common between the Suez and Panama Canals:

(1) Geomorphological Similarity.—Both isthmuses lie in the northern hemisphere and in approximately east and west direction. Hence, both canals run in nearly north and south direction. Both canals join with a sea on the north and a gulf on the south; Mediterranean Sea and Gulf of Suez in the case of the Suez Canal, Caribbean Sea and Golfe de Panamá in the case of the Panama Canal.

(2) Sea Level-Locks-Sea Level Sequence.—If the time scale for the Suez development is reduced and that of the Panama stretched, one readily sees that their historical unfoldings have revealed much in common in the order of

man's application of his ingenuity, despite that at present one is a sea-level canal and the other is a lock canal.

From the time of the Pharaohs of Egypt circa 1920 B.C. to the time of the Second Ptolemy in the early half of the third century B.C., the Suez Canal of the ancients had been without lock; the Second Ptolemy had a lock (maybe a diaphragm, but the Old English meant a lock) built. Even at the beginning of 1834, the Grand Council of Egypt still decided that the Delta locks should be constructed before the question of cutting through the Isthmus was considered. Until 1855, the Linant-Mougel plan was still with a lock at each end. Finally, the present sea-level canal was built in accordance with the 1859 plan.

The Panama Canal has shown the same course of man's thinking. In the 1870's and 1880's de Lesseps insisted that this one should be like the Suez ditch, at sea level, without locks. The commencement of excavation in 1882 was based on the sea-level plan.

After the United States bought the interests from the French company, "sea-level versus lock canal" has been at issue ever since periodically. A Board of Consulting Engineers was created on 24 June 1905 by President Theodore Roosevelt whose instructions in part stated:

> ". . . I hope that ultimately it will prove possible to build a sea-level canal. Such a canal would undoubtedly be best in the end, if feasible, and I feel that one of the chief advantages of the Panama route is that ultimately a sea-level canal will be a possibility. . . ."

On 10 June 1906, the Board of Consulting Engineers presented two reports: The majority report, signed by eight members, favored a sea-level canal; the minority report, signed by five members, favored a lock canal which was concurred in by the Isthmian Canal Commission and supported by its Chief Engineer, John F. Stevens, Hon. M. and Past-President, ASCE. An Act was then passed by Congress, requiring a lock canal, which became law by signature of President Roosevelt on 29 January 1907. The Panama Canal as constructed between 1904 and 1914 has six sets of locks.

The completion of the Panama Canal did not terminate the issue of "sea-level versus lock canal." A 12-volume report on the study of a proposed sea-level Panama Canal was prepared by the Corps of Engineers and printed in 1945. The Seventy-ninth Congress, First Session, expressed the temper of concern for the security of the canal by passing Public Law No. 280, which was approved by President Harry S. Truman on 28 December, 1945, and by which the Governor of the Panama Canal was authorized and directed to make a comprehensive review and study. Investigations under Public Law No. 280 disclosed that only an Isthmian sea-level canal will meet the future needs of interoceanic commerce and national defense. It resulted in Paper No. 2377 entitled "Sea Level Plan for Panama Canal," by J. G. Claybourn, F. ASCE, Trans. ASCE, Vol. 114, 1949, pp. 572-606; and Paper No. 2378 entitled "Panama Canal—The Sea-Level Project, A Symposium," ibid., pp. 607-906.

Thus, both canals have undergone the same sea level-locks-sea level sequence of considerations, except that in the case of the Suez Canal the time has stretched for nearly 4000 years, but in the case of the Panama Canal the time has elapsed for only about 80 years.

(3) British Intervention.—The United Kingdom exercised opposition and intervention during the early days of promotion in both cases. Her almost inexplicable opposition in the case of the Suez Canal when Lord Palmerston was Prime Minister may be found in George Edgar-Bonnet's book entitled

"Ferdinand de Lesseps—Le Diplomate—le créateur." In the case of the Panama Canal, the British attitude was fully reflected by the conclusion of the Clayton-Bulwer Treaty of 19 April 1850 and the Hay-Pauncefote Treaty of 5 February 1900.

(4) Neutralization.—The Suez Canal was neutralized by an accord between the Porte (Turkey) and eight European nations on 29 October 1888. Similar clauses of agreement were provided in the Hay-Pauncefote Treaty.

(5) Stepped Up Benefits to Egypt and Panama.—De Lesseps originally made provision in the Suez Canal Company's Statutes for the annual allowance to Egypt of 15% of the profits. In the 1936/7 Agreements, the yearly allocation by the Company to Egypt was increased to £E 300,000. Under the Convention signed on 7 March 1949, the Egyptian Government received a 7% share of the gross profits. This amounted to over £E 1,125,000 for the year 1951. In addition, the Company paid a whole series of taxes in Egypt, which in 1951 reached a total of nearly £E 3,000,000. On the eve of "nationalization," the Egyptian Government's share in profits and Egyptian taxes payable by the Company and by its security holders, amounted to no less than £E 4.4 million. There was also a provision of increasing employment of Egyptian personnel in the 1936/7 agreements; and in accordance with these agreements, Egyptian personnel received the same scale of salaries and wages and enjoyed the same employment benefits as the non-Egyptian personnel, save for the expatriation allowance.

In the case of the Panama Canal, as provided by the 1936 Treaty and the 1955 Treaty and Memorandum there has been the same trend of increasing annuity to Panama, establishing a single basic wage for Canal Zone agencies, extending the Civil Service Retirement Act to Panamanians, and affording equality of opportunity to Panamanians.

Major differences between the Suez and Panama Canals have been well covered by Governor Potter.

The 1956 Blockade and the 1957 Restoration

Mr. Shockley appropriately regards the restoration of the canal following the fighting in the later part of 1956 as ranking high in itself in engineering works. He thinks very valuable if an account thereof could be added. A brief narration was written of the November 1956 Blockage of the Waterway, the Clearance Operations from November 1956 to April 1957, and the Re-opening of the Canal on 24 April 1957. Due to words limit, all accounts subsequent to Nationalization were withheld for inclusion in a later paper.

General Wheeler, Consulting Engineer of the International Bank for Reconstruction and Development, formerly Chief of Engineers, U. S. Army, directed the operations in clearing the blockade under the auspices of the United Nations. The U. N. has not yet released a lengthy report. It is earnestly hoped that General Wheeler would some day write his authoritative first-hand account of the clearance of the blockade. He did a remarkable job at a crucial moment. Without a speedy clearance under the U. N., the pinch of the blockade on Western Europe's economic life would otherwise have been unconceivable.

The Array of Untreated Events Calling for a Later Paper

Space limitation in the Society's proceedings papers necessitated the confinement of the author's paper to the chronicle of important development and engineering events of the Suez Canal from its historical beginning to the

Egyptian Proclamation of Nationalization of 26 July 1956, leaving social, economic, and international aspects and what has occurred since that public declaration to a later paper.

Encouragements for its presentation have been enthusiastically expressed by Mr. Boillot, U. S. Representative of Compagnie Financière de Suez, an historian of the Canal; by General Wheeler, who was responsible for the clearing of the blockade; by Mr. Leslie, particularly in the economic aspects; by Dr. Chou, with emphasis in the strategic struggles staged to keep the Canal under control amidst the last two World Wars; and by Governor Potter, with interest in the aspects of the Canal since nationalization.

In such an international waterway enterprise as the Suez Canal, the broader social, economic, strategic, legal, and international aspects are of paramount importance in its successful development, maintenance, as well as operation. They are so intimately associated with such an interprise that engineering administration alone would not accomplish its mission should the broader aspects be not kept in sight.

The array of untreated events that would constitute the essential contents of a later paper should include:

1. Sublime Porte's Opposition against the Principle of Excavating the Canal, 1855-1856.
2. Palmerston's Opposition and British Intervention.
3. Disraeli's Deal of 1875.
4. Growth and Pattern of Canal Traffic, 1870 to 1955.
5. Reduction of Transit Dues, 1874 to 1954.
6. Egyptian Benefits since 1862.
7. British and French Benefits since 1869.
8. The Chronicle of Legal Aspects, 1854-1956.
9. The Neutralization Accord of 1888 and Its Sway.
10. The Canal during World War II.
11. Economic Life of the Isthmus since 1859.
12. Social Aspects of the Former Canal Company's Activities.
13. Piloting and Convoying of the Former Canal Company.
14. Egyptian-British Agreement on the Sudan in 1953 and Its Consequences.
15. The Nationalization of the Suez Canal Company on 26 July 1956.
16. Defense of the Suez Canal Company from the Morrow of 26th July 1956 to 16th May 1957.
17. The Joint Statement Issued in London on 3rd August 1956 by the United States, British and French Governments.
18. The London Suez Conference Held between 16 and 24 August 1956, and the Three Resolutions Adopted by the Majority at the London Conference on 22nd August 1956.
19. The Announcement of 12 September 1956, that the Suez Canal Users' Association Would Be Set Up.
20. The Second London Suez Conference Held between 19 and 21 September 1956.
21. Resolutions Passed by the United Nations Security Council on 13th October 1956, Containing 6 Requirements for Any Settlement of the Suez Question.
22. Intervention by Israel on 29th October 1956 and by France and the United Kingdom on 31 October 1956 in Egypt.

    (a) Consideration by the United Nations Security Council, 29-31 October 1956.

(b) Action by the General Assembly at the First Emergency Special Session, 31 October-10 November 1956.

(c) Non Compliance by the United Kingdom, France and Israel with the Cease-fire Resolution of 2 November 1956 of the General Assembly-Resolution 997 (ES-I): Request by USSR for Consideration by the Security Council on 5 November 1956.

23. The Blockade of the Suez Canal Waterway early in November, 1956.
24. The Pinch of the Suez Canal Blockade upon, and Emergency Oil Shipments to, Europe, End of November 1956 to April 1957.
25. Action by the United Nations General Assembly at Its Eleventh Session to Secure the Withdrawal of Israel, French and United Kingdom Forces from Egypt, November 1956-March 1957.

(a) Renewal Call by the General Assembly for the Withdrawal of Israel, French and United Kingdom Forces from Egypt-Resolution 1120 (XI) of 24 November 1956.

(b) Development of the United Nations Emergency Force (UNEF), 5 November 1956-March 1957.

(c) The Withdrawal of French, British and Israel Forces from Egypt, 27 November 1956-24 January 1957.

26. The Suez Canal Clearance Operations, November 1956-April 1957.
27. The Re-Opening of the Suez Canal on 24 April 1957.
28. Declaration on the Suez Canal and the Arrangement for Its Operation by Egypt on 24 April 1957, Embodying 10 Main Points.
29. The Transformation of the Suez Canal Company into Compagnie Financière de Suez, A Sociètè Anonyme au Capital de 10,747,750,000 Francs by Decree Dated 17th December 1957.
30. The Seeking of a Solution by the Egyptian Government to the Question of Compensation Due by Egypt as a Consequence of the Nationalization of the Canal, from the Visit to Cairo of the President of the International Bank for Reconstruction and Development on 9th November 1957 at the Invitation of the Egyptian Government, through three Conferences held in Rome, to the signing of the Voluntary Agreement on 29th April 1958, subject to Ratification.
31. International Bank for Reconstruction and Development Loan to Help Egypt Deepen Suez Canal in April 1958, and Dredging Operations following the Loan.
32. The Signing of the Agreement at Geneva, on 13th July 1958, between the Government of the United Arab Republic, Compagnie Financière de Suez (formerly known as Compagnie Universelle du Canal Maritime de Suez), and International Bank for Reconstruction and Development, in Respect of Compensation to Be Paid to the Company.
33. The Operation of the Suez Canal by Egyptian Government since the Re-Opening to traffic on 24 April 1957.

A presentation of the above 33 topics will complete the CHRONICLE and bring the events of the Suez Canal up to the present. The reception of the paper under closing discussion determines the later paper.

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Mr. Bostian poses two interrogatories, ". . . Where all or a large number of these references could be found," and "what percentage could be expected to be found in the library of the average engineering school?" To answer the former, it may be said that the Library of Congress, the Library of the City of New York, and the Engineering Societies Library in New York, in their combined holdings, contain practically all of those listed in the bibliography. In Europe, one may also find nearly all of them in the Bibliothèque nationale in Paris and in the Library of the British Museum in London. In engineering schools, library conditions vary greatly. However, the major items in the list of the bibliography may be found in the larger libraries of engineering colleges, and particularly in the library collection of the Rivers and Harbors Section, Department of Civil Engineering, Princeton University, Princeton, N. J., and that of Duke University, Durham, N. C.

Professor Morris mentions two references on the geology of the Canal route; Mr. Millecam reminds two remarkable papers dealing with protecting the Canal banks from erosion, and one book on Ferdinand de Lesseps; Mr. Leslie calls attention to the 1945 Corps of Engineers Report on the study of a proposed sea-level Panama Canal in 12 volumes in which there is interesting comparison of all major canals, including the Suez Canal; Mr. Blanquet refers to detailed report on model tests by Suquet-Barbier-Gamot, and the Ebasco Report; Mr. Bostian adds Encyclopaedia Britannica for a notable entry on the Suez Canal—all these 9 items together with 18 additional entries the writer has noted since the printing of the paper, are listed herein under items 107 to 133 as an extension of the Bibliography:

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114. Kirkpatrick, Ralph Z: Commerce Diversion-Suez to Panama; Civil Engineering, ASCE, Vol. 11, No. 6, June 1941, pp. 363 ff.

115. Report on the Study of A Proposed Sea-Level Panama Canal; in 12 Vols., 1945; Corps of Engineers, U. S. Army. (It contains interesting comparison of all major canals, including the Suez Canal.)

116. Bahon, Max: Report (Rapport) on Inland Navigation (Navigation Interieure); Section I-Communication 2, XVIIth International Navigation Congress, Lisbon, 1949, Permanent International Association of Navigation Congresses. (The arduous technical problem of protecting the banks of the Suez Canal from erosion caused by the passage of large vessels was treated.)

117. Suez Canal; The Encyclopedia Americana, 1949, Vol. 25, pp. 799-801; Americana Corporation, New York.

118. Edgar-Bonnet, George—Ferdinand de Lesseps—Le Diplomate-le Créateur de Suez; 1951; Printers: Plon Library; Editor: 8 rue Garancière, Paris VI.

119. Leroy, M; Report (Rapport) on Inland Navigation (Navigation Interieure); Section I-Communication 1, pp. 109-155, XVIIIth International Navigation Congress, Rome, 1953, Permanent International Association of Navigation Congresses. (The arduous technical problem of protecting the banks of the Suez Canal from erosion caused by the passage of large vessels was treated.)

120. Suez Canal; Encyclopaedia Britannica, 1953, Vol. 31, pp. 516-519; Encyclopaedia Britannica, Inc., London.

121. EBASCO Report; A report on Traffic Wishing to Use the Suez Canal up to 1972, to the Suez Canal Company, by Ebasco Services Incorporated, New York, 1957.

122. The Suez Canal Company and the Decision Taken by the Egyptian Government on 26th July 1956, Second Part (August 1956-May 1957); May 1957; Universal Company of the Suez Maritime Canal, Paris.

123. Suquet, Barbier et Gamot: Etude expérimentale des phénomènes accompagnant le transit des navires de fort tonnage dans le canal de Suez; Annales des Ponts et Chaussées; Vol. 127, November-December 1957, pp. 709-736, Vol. 128, January-February 1958, pp. 1-36.

124. Dollar Statement of Suez Canal Company Balance Sheet as of December 31, 1957 and Accounts for the Year 1957; The Universal Company of the Suez Maritime Canal, Paris.

125. Bulletin of the Suez Canal Company (English Edition); Bulletin de la Compagnie Universelle du canal Maritime de Suez (French Edition); Nos. 1 to 2339; The last three issues: No. 2337, January 1958; No. 2338, Février 1958; No. 2339, May 1958, contain full information on the final status of the former Suez Canal Company; The Universal Company of the Suez Maritime Canal, Paris.

126. International Bank for Reconstruction and Development Loan to Help Egypt Deepen Suez Canal; Engineering News-Record, Vol. 160, April 17, 1958, p. 94.

127. Bulletin, Compagnie Financière de Suez; No. 1, July 1958, and succeeding issues; Compagnie Financiere de Suez, Paris. (The settlement reached between the Government of the United Arab Republic and the Universal Company of the Suez Maritime Canal is described in detail in No. 1.)

128. Americans Low Bid Suez Canal Dredging; Engineering News-Record, Vol. 161, No. 3, July 17, 1958, p. 60.

129. Manhattan Dredger Writes Chapter I in New Suez Saga; Investor's Reader, July 23, 1958, pp. 12-14; Merrill Lynch, Pierce, Fenner & Beane, New York.

130. Suez Award Holds Surprises; Engineering News-Record, Vol. 161, No. 6, August 7, 1958, p. 26.

131. Schoon, H. F.: Suez Dredging Holds More Surprises (letted to the editor); Engineering News-Record, Vol. 161, No. 13, September 25, 1958, p. 12.

132. U. S. Dredge Off to Suez Canal; Engineering News-Record, Vol. 162, No. 2, January 15, 1959, p. 25.

133. Giant Hydraulic Dredge Is Off For Work in Egypt; Civil Engineering, ASCE, Vol. 29, No. 4, April 1959, p. 108 (Vol. P. 298).

This humble effort to document and list the available published works, records, and literature relating to the Suez Canal has no comparison with the "approximately 7,000 items" the Canal Zone Government Library has cataloged "on the Republic of Panama and the Panama Canal" as cited by Governor Potter. Despite that a complete bibliography on Egypt and the Suez Canal would outnumber that of Panama, it should be recognized that published works on the Suez Canal is comparatively scarce due to (1) its earlier construction by about half a century in a period publication was not as easy as the twentieth century, (2) less construction problem at Suez as at Panama, (3) private instead of government enterprise at Suez, and (4) the compilation having been done on this side of the ocean.

Typographical Corrections

Mr. Millecam as well as the author noted that on Page 1770-8, line 14, the year "1958" should read "1858". And on Page 1770-11, line 19, the author noted the word "salaries" should read "salaried."

Postlude

In closing, the author wishes to thank all of the discussers for their enthusiasm in contributing so much valuable comments. It has been his greatest pleasure to carefully read the discussions and to ponder further on the subject while writing this closure. If the paper has supplied a need for authenticating the Suez Canal events, has served a framework for further study, and has provoked any historical interest on the subject, the author will feel more than duly rewarded. Let the closure be closed by quoting:

> "I shall be content if those shall pronounce my history useful who dèsire to give a view of events as they really happen."
>
> -Thucydides, Athenian Historian;
> 471 ? - 400? B.C.

"He who considers things in their first growth and origin—will obtain the clearest view of them."

-Aristotle, Greek Philosopher; 384-322 B.C.

NAVIGATION ON THE COLUMBIA RIVER[a]

Closure by Ray E. Holmes

RAY E. HOLMES,[1] F. ASCE.—Mr. Hickson, through his long experience on the Columbia River, contributes several items of additional information on the subject for which the author is grateful.

Since publication of the report, improvements for navigation have occurred which should provide additional stimuli to the growth of navigation on the river. Pinnacle removal has been completed in Bonneville pool, and the greater part of the channel between Bonneville Dam and Vancouver has been restored to a 27-foot depth. The balance of the work is scheduled for completion early in 1960 and will make available a deep-draft channel to The Dalles, Oregon, 190 miles inland from the Pacific Ocean.

Movement of alumina from the Orient directly to an aluminum plant at The Dalles is expected as soon as the channel is completed. Movement of other commodities such as products of agriculture and mining should follow.

a. Proc. Paper 1789, September, 1958, by Ray E. Holmes.
1. Chf., Rivers & Harbors Section, Portland Dist., Corps of Engrs., U. S. Dept. of the Army, Portland, Ore.

COLUMBIA BASIN STREAMFLOW ROUTING BY COMPUTER[a]

Closure by David M. Rockwood

DAVID M. ROCKWOOD,[1] A. M. ASCE.—The author wishes to thank Mr. Snyder for the contribution of his discussion on routing methods. The routing procedure he describes is similar to Tatum's method of "Successive Averages," as set forth in Linsley, Kohler and Paulhus' "Applied Hydrology"(1) except that the coefficients are derived by statistical weightings rather than by an arbitrary weighting system. With regard to Mr. Snyder's system, the method of statistical weighting may give unrealistic values of distribution coefficients. This is illustrated by the values of the ordinates for the function shown on Fig. A-3 of Mr. Willard's discussion. Here, the sum of the ordinates at the end of each 4-hour period is approximately 1.35. From a strictly rational point of view, the sum of the ordinates should equal unity. The derived unit distribution graph shown on Fig. A-2 for the local area inflow has a volume of about 0.31 inches rather than the theoretical value of one inch. For this particular flood event as analyzed by Mr. Snyder, it appears that the statistical procedure "over-weighted" the channel routing coefficients, and "under-weighted" the local inflow coefficients, in order to provide a best fit of the data as a whole. It would seem particularly desirable to have the sum of the channel routing coefficients equal to unity. For the local inflow, the derived coefficients may reflect differences between assumed precipitation and loss values, and the true amounts.

In closing, the author wishes to generalize briefly on streamflow routing procedures. There are almost as many and varied routing procedures as there are hydrologists who use them. The methods range from a very simple and empirical approach, to a highly complex theoretical analysis of the hydraulics of non-steady flows in open channels. In any case, it must be remembered that the basic ingredient is hydrologic data whose accuracy will always involve errors of measurement and approximations of estimates. The engineer who is confronted with the selection of a routing method should consider the problem as a whole, including the type of channel, type of study, the effect of varying backwater conditions, basic requirements of accuracy, time for solution, relative importance of results, and the resources available for computation.

In the case of the procedure outlined by the author for the Columbia River Basin, the prime requirements were speed of operation, ability to adjust routings and forecasts on the basis of observed streamflow conditions from day-to-day, computer drum storage space limitations for storage of routing coefficients (rather than unit hydrograph ordinates), and the practical limitations

a. Proc. Paper 1874, December, 1958, by David M. Rockwood.

1. Hydraulic Engineer, U. S. Army Engrs., Portland, Ore.

that streamflow forecasts should be obtainable within three to four hours, from receipt of basic data. This includes the time for processing data, adjusting routing values to observed streamflow conditions, tabulating and keypunching input values of streamflow and forecasted snowmelt, and the actual running time on the computer. The routing procedure selected is one which is made possible by use of a medium-speed electronic digital computer. The procedure is intermediate in the scale between an entirely empirical method and a highly complex theoretical approach. The concept of the use of time of storage, T_s, is fundamental to this procedure. The fact that T_s can be computed directly for reservoirs of known storage and outlet characteristics enables the engineer to evaluate explicitly the inflow-outflow relationships for reservoirs. The application of the technique to evaluating storage effects in river channels by successive short reaches (or phases) of reservoir-type storage has been demonstrated. There is a physical relationship for open channels between time of storage, T_s, and the storage and flow characteristics in a given reach, which may be evaluated directly for reaches where these conditions are known by use of Eq. (5) in the original paper. The trial and error procedure used in evaluating channel routing coefficients is actually a matter of verifying or refining values derived mathematically from generalized streamflow and channel storage data. The procedure has the additional provision for varying the time of storage (T_s) with discharge; this is realistic, inasmuch as the travel time of flood waves is known to vary with the flow condition. Any function may be inserted to account for this variation. It has also been shown that successive routings through reservoir-type storage can be used to distribute runoff from rainfall or snowmelt, and by varying the routing coefficients, any desired shape of distribution may be obtained.

The procedure has been used successfully for forecasting streamflow in the Columbia River Basin during the flood season for three years. For each year, forecasts were made each day for 30 to 35 consecutive days. Results have shown it to be entirely feasible, and the principal errors in verification of forecasts were due to unforeseen weather conditions. In addition to streamflow forecasting, the procedure has been used in the derivation of the maximum probable flood for the Salmon River Basin, which drains about 14,100 Sq. Mi. in Central Idaho; evaluating the effect of loss of valley storage in the Priest Rapids-Wanapum reach of the Columbia River, located just upstream from the mouth of Snake River; and for general hydrologic studies of snowmelt and runoff conditions for each of 21 sub-basins in the Columbia Basin.

REFERENCE

1. Linsley, R. K., Jr., Max A. Kohler, and Joseph L. H. Paulhus, Applied Hydrology, McGraw-Hill, New York, 1949.

GENERALITIES ON COASTAL PROCESSES AND PROTECTION[a]

Discussion by R. Silvester

R. SILVESTER.[1]—The author is to be commended for drawing attention to the ineffectiveness of groins. The writer has recently come to the same conclusion(1) although he would not support wholeheartedly the alternative submitted, that of artificial nourishment. He would prefer to see the offshore breakwater used more widely. These, floated into position and sunk in echelon fashion, could impede sediment movement over a wide area of the ocean bed. They could even be removable should changing conditions demand. Of course, the same problems of down-coast erosion would arise, but this can be removed to an undeveloped section of coast or the end of a physiographic unit.

The author states "It is clear that in the absence of a resulting lateral transport groins or other structures designed for or having the effect of impeding such transport have no meaning. On the other hand they are also rather harmless." Although the situation is rather academic, the writer would disagree with the latter sentence because storm conditions could sponsor rip currents more readily in the moon-shaped beaches between the groins than on a plane beach. These currents throw the sand out of reach of the waves or help it to by-pass the groin system.

The author mentions some permanent erosive conditions. Perhaps another, in general not fully appreciated, is that occurring at the updrift end of a physiographic unit. Such a situation is a sharp change of direction in a coastline where sediment is being removed from a "cape". As time goes by the littoral drift diminishes as the sand nearest the cape is removed and the waves find it harder to remove the remainder. Downdrift of the cape, where once there was a substantial littoral drift, becomes "under-nourished". The effect of this may not be exhibited on the foreshore for many years.

The author states "An important point in favour of artificial supply is the fact that it does not entail a permanent commitment. It can do no harm." It might be said that groins also may not be a permanent commitment. They can be allowed to silt up or to be undermined like any natural rock outcrop. Certainly reducing them in size is an uneconomical proposition. Artificial nourishment, on the other hand, may do harm by silting up a harbor entrance down drift of the area. The author would presume that such mismanagement could not occur.

The author states "A structure designed as an impediment to wave-induced drift, cannot serve to control the flow in a tidal channel that may have depths of 10 to 20 m. or even more." This seems to imply that sand cannot be

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a. Proc. Paper 1976, March, 1959, by J. B. Schijf.

1. Senior Lecturer in Civ. Eng. (Hydraulics), University of Western Australia, Nedlands, Australia.

transported in these depths unless currents exist which are not associated with wave action. Evidence is available from nature and scaled models to show that sand can be moved in such depths solely by wave action. The mass-transport "current" and other phenomena associated with the oscillatory motion of the water particles at the ocean bed effect a considerable movement. This all points to the fact that groins have little influence on the total width of ocean bed over which coastal sediment movement takes place.

REFERENCE

1. R. Silvester, "Engineering Aspects of Coastal Sediment Movement", Am. Soc. Civil Engrs., Proc. Paper 2168, September, 1959.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division during 1958.

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SEPTEMBER: 1750(IR3), 1751(IR3), 1752(IR3), 1753(IR3), 1754(IR3), 1755(ST5), 1756(ST5), 1757(ST5), 1758(ST5), 1759(ST5), 1760(ST5), 1761(ST5), 1762(ST5), 1763(ST5), 1764(ST5), 1765(WW4), 1766(WW4), 1767(WW4), 1768(WW4), 1769(WW4), 1770(WW4), 1771(WW4), 1772(WW4), 1773(WW4), 1774(IR3), 1775(IR3), 1776(SA5), 1777(SA5), 1778(SA5), 1779(SA5), 1780(SA5), 1781(WW4), 1782(SA5), 1783(SA5), 1784(IR3)[c], 1785(WW4)[c], 1786(SA5)[c], 1787(ST5)[c], 1788(IR3), 1789(WW4).

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JANUARY: 1892(AT1), 1893(AT1), 1894(EM1), 1895(EM1), 1896(EM1), 1897(EM1), 1898(EM1), 1899(HW1), 1900(HW1), 1901(HY1), 1902(HY1), 1903(HY1), 1904(HY1), 1905(PL1), 1906(PL1), 1907(PL1), 1908(PL1), 1909(ST1), 1910(ST1), 1911(ST1), 1912(ST1), 1913(ST1), 1914(ST1), 1915(ST1), 1916(AT1)[c], 1917(EM1)[c], 1918(HW1)[c], 1919(HY1)[c], 1920(PL1)[c], 1921(SA1)[c], 1922(ST1)[c], 1923(EM1), 1924(HW1), 1925(HW1), 1926(PL1), 1927(HW1), 1928(HW1), 1929(SA1), 1930(SA1), 1931(SA1), 1932(SA1).

FEBRUARY: 1933(HY2), 1934(HY2), 1935(HY2), 1936(SM1), 1937(SM1), 1938(ST2), 1939(ST2), 1940(ST2), 1941(ST2), 1942(ST2), 1943(ST2), 1944(ST2), 1945(HY2), 1946(PO1), 1947(PO1), 1948(PO1), 1949(PO1), 1950(HY2)[c], 1951(SM1)[c], 1952(ST2)[c], 1953(PO1)[c], 1954(CO1), 1955(CO1), 1956(CO1), 1957(CO1), 1958(CO1), 1959(CO1).

MARCH: 1960(HY3), 1961(HY3), 1962(HY3), 1963(IR1), 1964(IR1), 1965(IR1), 1966(IR1), 1967(SA2), 1968(SA2), 1969(ST3), 1970(ST3), 1971(ST3), 1972(ST3), 1973(ST3), 1974(ST3), 1975(ST3), 1976(WW1), 1977(WW1), 1978(WW1), 1979(WW1), 1980(WW1), 1981(WW1), 1982(WW1), 1983(WW1), 1984(SA2), 1985(SA2)[c], 1986(IR1)[c], 1987(WW1)[c], 1988(ST3)[c], 1989(HY3)[c].

APRIL: 1990(EM2), 1991(EM2), 1992(EM2), 1993(HW2), 1994(HY4), 1995(HY4), 1996(HY4), 1997(HY4), 1998(SM2), 1999(SM2), 2000(SM2), 2001(SM2), 2002(ST4), 2003(ST4), 2004(ST4), 2005(ST4), 2006(PO2), 2007(HW2)[c], 2008(EM2)[c], 2009(ST4)[c], 2010(SM2)[c], 2011(SM2)[c], 2012(HY4)[c], 2013(PO2)[c].

MAY: 2014(AT2), 2015(AT2), 2016(AT2), 2017(HY5), 2018(HY5), 2019(HY5), 2020(HY5), 2021(HY5), 2022(HY5), 2023(PL2), 2024(PL2), 2025(PL2), 2026(PP1), 2027(PP1), 2028(PP1), 2029(PP1), 2030(SA3), 2031(SA3), 2032(SA3), 2033(SA3), 2034(ST5), 2035(ST5), 2036(ST5), 2037(ST5), 2038(PL2), 2039(PL2), 2040(AT2)[c], 2041(PL2)[c], 2042(PP1)[c], 2043(ST5)[c], 2044(SA3)[c], 2045(HY5)[c], 2046(PP1), 2047(PP1).

JUNE: 2048(CP1), 2049(CP1), 2050(CP1), 2051(CP1), 2052(CP1), 2053(CP1), 2054(CP1), 2055(CP1), 2056(HY6), 2057(HY6), 2058(HY6), 2059(IR2), 2060(IR2), 2061(PO3), 2062(SM3), 2063(SM3), 2064(SM3), 2065(ST6), 2066(WW2), 2067(WW2), 2068(WW2), 2069(WW2), 2070(WW2), 2071(WW2), 2072(CP1)[c], 2073(IR2)[c], 2074(PO3)[c], 2075(ST6)[c], 2076(HY6)[c], 2077(SM3)[c], 2078(WW2)[c].

JULY: 2079(HY7), 2080(HY7), 2081(HY7), 2082(HY7), 2083(HY7), 2084(HY7), 2085(HY7), 2086(SA4), 2087(SA4), 2088(SA4), 2089(SA4), 2090(SA4), 2091(EM3), 2092(EM3), 2093(EM3), 2094(EM3), 2095(EM3), 2096(EM3), 2097(HY7)[c], 2098(SA4)[c], 2099(EM3)[c], 2100(AT3), 2101(AT3), 2102(AT3), 2103(AT3), 2104(AT3), 2105(AT3), 2106(AT3), 2107(AT3), [illegible], 2111(AT3), 2112(AT3), 2113(AT3), 2114(AT3), 2115(AT3), 2116(AT3), 2117(AT3), 2118(AT3), 2119(AT3), [illegible], 2121(AT3), 2122(AT3), 2123(AT3), 2124(AT3), 2125([illegible]).

AUGUST: 2126(HY8), 2127(HY8), 2128(HY8), 2129(HY8), 2130(PO4), 2131(PO4), 2132(PO4), 2133(PO4), 2134(SM4), 2135(SM4), 2136(SM4), 2137(SM4), 2138(HY8)[c], 2139(PO4)[c], 2140(SM4)[c].

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c. Discussion of several papers, grouped by divisions.

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NO. WW 3
PART 2

Your attention is invited

NEWS OF THE WATERWAYS AND HARBORS DIVISION OF ASCE

AMERICAN SOCIETY OF CIVIL ENGINEERS
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JOURNAL OF THE WATERWAYS AND HARBORS DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

DIVISION ACTIVITIES
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

NEWS

September, 1959

CONVENTION PLANS

With the Washington Convention less than one month away, it is interesting to review the work of the Waterways and Harbors Division Committee on Session Programs, which arranges all the technical meetings of the Division.

While the Los Angeles and Cleveland Conventions may be considered "water over the dam", they should be more aptly classified as "feathers in the Committee's cap." Colonel Carroll T. Newton and Richard M. Gensert, the respective contact members, did such a splendid job in arranging details and assisting the chairmen of our Technical Committees in the presentation of our programs. Both of these programs were most enthusiastically received.

Capt. L. E. Root, CEC, USN, contact member for the Washington Convention, furnished so many suggestions for papers that it was difficult to choose those for inclusion in the four sessions allotted our Division. The Coastal Engineering Committee and the Committee on Flood Control and Navigation Facilities will sponsor one session each and the Committee on Ports and Harbors will sponsor the remaining two sessions. The papers are being presented by outstanding engineers on most interesting subjects. So, reserve both October 19 and 20 for Waterways and Harbors Division sessions if you are planning to attend the Washington Convention.

Moving on to the New Orleans Convention in March 1960, William (Bill) C. Cobb got off to such a flying start as contact member that it was necessary to request and we have been allotted five sessions for this meeting. At the present writing, the program is being reviewed by the Division Executive Committee. If all goes as planned, every Technical Committee of the Waterways and Harbors Division will be represented by sponsoring one or more papers. Tentative arrangements have also been made to have the Construction Division join us in several of the sessions.

Reaching further into the future, the Committee has been fortunate in obtaining Harry A. Dettmer as our contact member for the Reno Convention in

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June 1960. He has worked up a comprehensive list of possible subjects and authors covering projects on the West Coast. This list has been furnished the Chairmen of the respective Technical Committees for consideration in developing the program which they may wish to sponsor for this meeting. We have been tentatively allotted four sessions for this meeting.

Brig. Gen. Alden K. Sibley, Division Engineer, U. S. Army, New England Division, has been appointed "contact member" for the Boston Convention in October 1960. The Committee has requested allotment of four sessions for this meeting and can confidently look forward to an outstanding program.

REGULATION OF COASTAL STRUCTURES

Mr. Herbert C. Gee, Chairman of the Task Committee on Regulation of Coastal Structures of The Division Committee on Coastal Engineering, reports that his newly formed group has contacted each of the 30 governors of states having coastal frontage on the Atlantic Ocean, the Gulf of Mexico, the Pacific Ocean or the Great Lakes. In each instance the governor was asked to submit to the committee, information concerning the law in his own state as it pertains to regulation of structures in tidal waters or in the waters of the Great Lakes.

To date, full information has been received from 17 states. Ten additional states have acknowledged receipt of the Task Force letter and referred the matter to the Attorney General. Three states have not replied and a follow-up letter has been written to these three governors.

The Committee is hopeful that all replies will be in hand by September so that work can begin on the compilation of data received.

NEWS OF COMMITTEE MEMBERS

Mr. Lester W. Angell, a member of the Division Committee on Navigation and Flood Control Facilities, now heads the engineering staff for the U. S. Study Commission, Southeast River Basins, and is located in Atlanta, Georgia. The Commission is an independent Federal agency charged with making a comprehensive study on the land and water resources of river basins in Alabama, Florida, Georgia, and South Carolina. Mr. Angell was formerly Director of Engineering Design, Planning and Program for the St. Lawrence Seaway Development Corporation.

Mr. Thomas J. Fratar, a member of the Division Committee on Ports and Harbors, was recently elected a member of the Engineering Foundation Board. Mr. Fratar will fill the unexpired term of the late Leslie G. Holleran and will serve until May, 1963. The Engineering Foundation is the research organization of United Engineering Trustees, Inc., which is composed of the ASCE, ASME, AIEE, AICHE and AIMMPE. Mr. Fratar is a director of ASCE and a partner in Tippetts-Abbett-McCarthy-Stratton.

EXPANSION OF LOCAL SECTION ACTIVITIES

Francis G. Christian, Chairman of The Division Committee on Cooperation with Local Section, and the members of the control group of the Committee have formulated operating procedures for the control group and for the

Local Section Representatives. The objectives of establishing these operating procedures are as follows:

1. To interest ASCE members in affiliating with the Waterways and Harbors Division;
2. To bring the activities of the Waterways and Harbors Division to the attention of local sections and to encourage local section presentation of waterways and harbors subjects;
3. To publicize the activities of the Committee; and
4. To improve the operations of the Committee. Under the new operating procedures, the Committee should further expand the Division's activities on the local level.

The Committee members are as follows:

Clifton T. Barker, Tenn. Valley Section
Francis G. Christian, Sacramento Section
Joseph B. Converse, Alabama Section
Walter F. Lawlor, St. Louis Section
Richard F. Lyman, Jr., San Francisco Section
N. C. Magnuson, No. Carolina Section
Prof. Paul G. Mayer, Ithaca Section
Henry O. Mikelait, Jr., Miami Section
Floyd D. Peterson, Natl. Capital Section
Howard A. Preston, Columbia Section
Geroge A. Smith, Indiana Section
J. Thornton Starr, Maryland Section
Harvill E. Weller, Mid-South Section
Basil W. Wilson, Texas Section
Alfred C. Winters, Oklahoma Section

NEW COMMITTEE MEMBERS

Ben E. Nutter, Chief Engineer of the Port of Oakland and Chairman of The Division Committee on Ports and Harbors, reports that William T. Hogg, Director of Engineering for The Port of New Orleans, and Richard G. Krahn, Harbor Engineer for Milwaukee, have been appointed to the Committee. These new members further broaden the Committee representation.

Evan W. Vaughan of Parsons, Brinkerhoff, Hall and MacDonald will become a member of the Division Executive Committee in October. Robert J. Winters, Cooridinator of Engineering for The Port of New York Authority, takes over Mr. Vaughan's duties as Secretary of the Division.

CORRECTION

In the June issue of The Waterways and Harbors Division Newsletter, the appointment of Mr. Thorndike Saville, Jr., as Chairman of The Division Committee on Coastal Engineering was announced. Mr. Saville, Jr., is the Assistant Chief, Research Division, Beach Erosion Board, Corps of Engineers in Washington, D. C. Unfortunately, the Newsletter listed Mr. Saville, Jr., with his father's title and address. The senior Mr. Thorndike Saville is

Director, Science and Engineering Center Study, University of Florida at Gainesville, Fla. We regret any inconvenience this unfortunate confusion between father and son may have caused.

FOR YOUR CALENDAR

October 19-23, 1959	ASCE, Washington, D. C. Convention
January 19-20, 1960	Princeton Conference
March 7-11, 1960	ASCE, New Orleans Convention
June 19-23, 1960	ASCE, Reno Convention
October 9-13, 1960	ASCE, Boston Convention
April 10-15, 1961	ASCE, Phoenix Convention
October 16-20, 1961	ASCE, New York Convention
February, 1962	ASCE, Houston Convention
May, 1962	ASCE, Omaha Convention
October 15-19, 1962	ASCE, Detroit Convention

NEWSLETTER PUBLICATION

The next issue of The Waterways and Harbors Division Journal will be in December, 1959. The deadline for submission of copy for that issue is October 31, 1959. If you have any material that might be usable in the Newsletter which will accompany the Journal, please send it to:

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